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APPLICATION OF TRANSVERSE PRESTRESSING FOR BRIDGE DECKS

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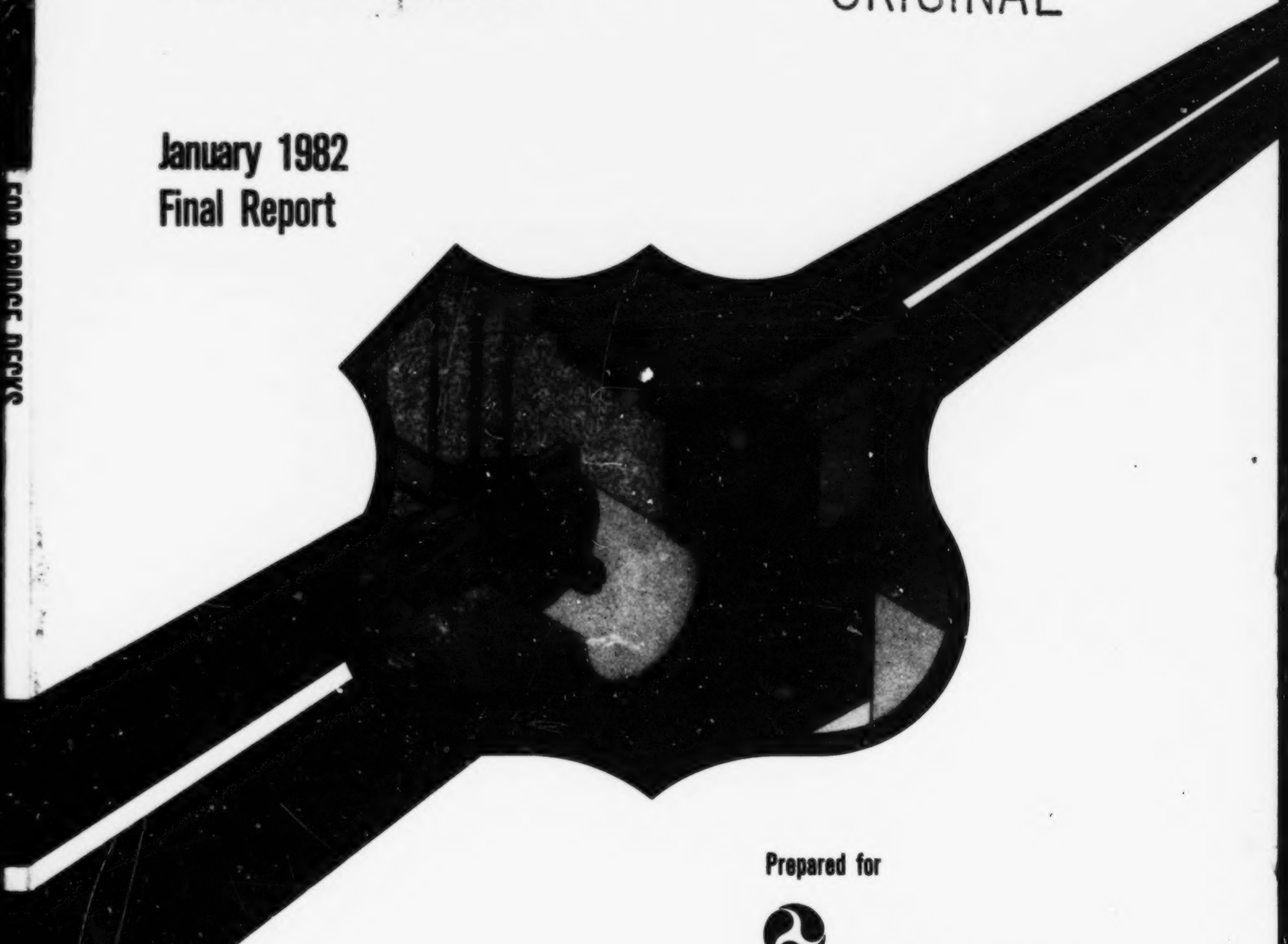

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I

FOREWORD

This report presents the results of a study which addresses current and proposed uses of transverse prestressing in concrete bridge decks. The report will be of interest to engineers involved in the design, analysis, construction and rehabilitation of highway bridges.

The research documented in this report was done as part of the Federal Highway Administration FCP Program. Results are being integrated into FCP Project 5K entitled "New Bridge Design Concepts." Mr. Craig A. Ballinger is the project manager and Mr. Thomas Krylowski is the task manager.

Sufficient copies of the report are being distributed to provide a minimum of one copy to each regional office, one copy to each division office, and two copies to each State highway department. Direct distribution is being made to the division offices.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

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16. Abstract

Previous research and utilization of transverse prestressed elements was reviewed for the purpose of developing a state-of-the-art report on the concept of transverse prestressing of bridge decks. This material served as the basis for panel discussions among senior engineers to develop new and innovative concepts for design of prestressed decks.

The concepts were developed into four design schemes utilizing transversely and longitudinally prestressed precast deck slab modules. Two schemes were prepared for application in bridge deck rehabilitation. The remaining two dealt with precast prestressed deck modules for new construction. Plans, details and design computations were developed for each in sufficient detail to describe the manner in which each would be constructed.

A cost-benefit analysis was completed for each concept, comparing it with a conventional cast-in-place deck slab.

The Final Report includes a state-of-the-art report and concept design criteria and details. Plans, cost estimates and design computations are given for each of the four design concepts.

17. Key Words

Bridge rehabilitation, increased efficiency of bridge decks, bridge deck improvements, modular bridge deck units, prestressed modular bridge deck elements

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TABLE OF CONTENTS

Page

I INTRODUCTION 1

- Scope 1
- Potential Benefits 1
- Study Objectives 3
- Research Approach 3

II STATE OF THE ART 4

- Literature Search 4
- Agency Contacts 4
- Detailed Investigations 5

III DESIGN CONSIDERATIONS 19

- Design Criteria 19
- Design Details 21

IV DESIGN CONCEPTS

- General Discussion 37
- Design Concept Number 1 - Bridge Deck Replacement 38
- Design Concept Number 2 - Bridge Rehabilitation 47
- Design Concept Number 3 - New Bridge Construction 55
- Design Concept Number 4 - New Bridge Construction 60

V COST BENEFIT ANALYSIS 67

- Factors Considered 67
- Analysis 71
- Cost Benefit Summary 73

VI CONCLUSIONS 75

REFERENCES 77

APPENDICES

A Structural Computations
B Cost Benefit Analysis

LIST OF FIGURES

Number	Title	Page
1	Roadway Elevation of Indiana Route 37 Bridge over Bean Blossom Creek	6
2	Typical Spring Clip Attachment Between Deck Slab and Support Girder	7
3	Typical Joint on Roadway Surface	9
4	View of End of Slab Section at Curb	9
5	Roadway Elevation of Bridge Carrying Indiana Route 140 over the Big Blue River	10
6	View of Roadway Surface - Route 140 Bridge	12
7	Schematic of Fiberboard Bearing Material and Clamping Bolt. Tonkel Road Bridge	13
8	Schematic View of Weld Plate for Shear Resistance. Tonkel Road Bridge	13
9	Double Female Type Joint	14
10	Commonly Used Shear Key Configurations	23
11	Typical Lap Type Longitudinal Joint	24
12	Schematic Showing Need to Provide Sufficient Strand Development Length to Assure Full Capacity Prior to Entering an Area of Maximum Moment	25
13	Steel Plate Embedded in Precast Girder	28
14	Prestressed Concrete Beam Stirrup Arrangement	28
15	Shear Plate Attached to Concrete Beam	29
16	Stud Welding to Shear Plate	30
17	Precast Slab Bolted to Concrete	31
18	Shim Pads Under Slab	34
19	Steel Angle Support System	34
20	Bolt Support	35
21	Typical Cross-Section - Design Concept Number 1	39
22	Concept No. 1 - Bridge Rehabilitation	41

(LIST OF FIGURES (Cont.)

Number	Title	Page
23	Typical Cross-Section - Design Concept Number 2	49
24	Concept No. 2 - Bridge Rehabilitation	50
25	Typical Cross-Section - Design Concept Number 3	56
26	Concept No. 3 - New Bridge	57
27	Typical Cross-Section - Design Concept Number 4	61
28	Concept No. 4 - New Bridge	62

LIST OF TABLES

Number	Title	Page
1	Production Rates for Cast-in-Place Construction Activities	68
2	Comparison of Materials Handling Used in Precast Concrete Construction and Cast-in-Place Construction	69
3	Summary of Cost Benefit Analysis	74

CHAPTER I. INTRODUCTION

SCOPE

Current practice in the design of cast-in-place reinforced concrete bridge decks is based upon the requirements of stiffness, lateral and longitudinal distribution of loads, and placement of reinforcement. The requirement for adequate concrete cover over top reinforcement to prevent the deleterious effects of deicing chemicals has made it necessary to design a structurally inefficient deck slab in relation to the relative strengths of the concrete and steel of which it is composed.

One alternative to the conventional deck design is the use of transverse prestressed concrete deck slabs. Slabs can be either cast-in-place and post-tensioned or can be precast and prestressed. The system is adaptable to new construction and to deck replacement projects. Under the scope of this study, prestressing with cast-in-place construction was not included.

POTENTIAL BENEFITS

When compared with conventional construction techniques, transverse prestressing, coupled with precasting, offers several potential benefits, including better quality control, reduced maintenance requirements, shorter down time for deck replacements, greater economy, and improved structural efficiency. Post-tensioning cast-in-place construction may provide some of these benefits.

Quality Control

By casting concrete in a controlled environment, it is easier to maintain uniformity and control in the quality of the end product. Precasting plants, especially when those plants are under roof, permit greater control over curing procedures and temperature. In many cases, a permanent concrete mixing operation will provide greater uniformity in the concrete than field batch plants.

Maintenance Needs

Through quality control, better performance can be expected and therefore reduced maintenance. Precast structural elements in bridge construction have proven to be more durable and less susceptible to deterioration from adverse environmental conditions, based on discussions with maintenance engineers from several state highway departments and inspection of several bridges as a part of this project.

Construction Time for Replacement Decks

The use of precast deck panels can reduce the time needed for on-site construction work significantly. Under heavy traffic where inconvenience to motorists is a major concern, precasting may save both time and dollars.

Structural Efficiency

By either pretensioning or post-tensioning the deck slab, greater spacing of girders can be accommodated with the same depth slab. Also, increased cantilever lengths at the fascia beams can be handled. Greater spacing of beams and girders results in fewer members and therefore reduced dead load. This in turn permits lighter substructure units to be used. The net result is a lighter substructure and superstructure.

Economy

While the cost of a deck utilizing pretensioning may be higher than a deck constructed utilizing conventional techniques, savings may be realized in other structural elements or construction procedures. In new bridge construction, creating a more efficient structural system as discussed above may result in a more economical superstructure and substructure. In replacing decks on existing structures, precast prestressed sections may result in a reduced construction period and less traffic control effort. A detailed analysis of costs is included in a later section of the report.

STUDY OBJECTIVES

The objectives of the study were to develop several concepts for design, analysis and construction details utilizing transverse prestressing, to develop four design examples and example plans, and to evaluate the concepts through cost benefit analyses.

RESEARCH APPROACH

To achieve the objectives of the study several steps were taken. First, a comprehensive literature search was conducted. This was followed by field inspections and discussions with designers and contractors who had experience in working with transverse prestress concepts in Pennsylvania, New York and Indiana.

A panel of senior engineers from Byrd, Tallamy, MacDonald and Lewis, T. Y. Lin International and the Federal Highway Administration was convened to consider alternative and innovative concepts for the design of bridge decks using transverse prestressing. The panel met on two separate occasions. The first was to investigate a wide variety of ideas brought to the meeting by individual panel members. To assist them in preparing for this meeting, the results of the literature search and the field investigations were provided for background.

At the second meeting the panel reviewed numerous concepts. Four of these were recommended for further refinement and the preparation of example calculations and construction drawings.

CHAPTER II. STATE OF THE ART

LITERATURE SEARCH

A literature search was performed, using an in-house library, the FHWA library, the TRB library and a computer information search by the Highway Research Information Service.

Summary reports of those documents pertinent to the study were prepared with a subject listing so that data could be recalled when needed.

Material found in the search dealt with the use of precast concrete form panels that serve the dual purpose of producing a better form for the cast-in-place portion of the slab and also act compositely with the cast-in-place portion of the slab to resist imposed loads. Other documents pertained to segmental construction wherein the deck and the supporting systems are monolithically precast. Because of the work already completed by others in researching these deck panel forms and their widespread usage, it was decided to eliminate this concept from the study.

Several reports were developed on experience in using precast concrete deck systems but using mild reinforcement rather than either post-tensioning or prestressing in the transverse direction. Some of the details developed with those systems can be used with a prestressed system and will be discussed in a later section of this report.

Two projects had significant relevance to the study--research done by Purdue University involving the use of transverse prestressing on two bridges in the state of Indiana and projects done by the Atcheson, Topeka & Santa Fe Railway Company using a similar system. These are described in detail later in the report.

AGENCY CONTACTS

Meetings were held during the early stages of the project with representatives of highway departments, universities, and private contractors. The purpose of these meetings was to obtain additional information about the design, fabrication and service life of precast slabs.

The states of Indiana, Pennsylvania, and Texas, the New York State Thruway Authority, Purdue University, Penn State University and the Atcheson, Topeka and Santa Fe Railway provided valuable information on precast bridge deck design and construction.

Three precast concrete fabricators were interviewed to provide input concerning fabrication practices and problems.

DETAILED INVESTIGATIONS

Several structures were inspected by the research team which employed full or partial depth precast concrete deck slabs. These included two bridges in the state of Indiana, on on the New York State Thruway, and one in the state of Texas. In addition to field visits, plans and specifications relative to other bridges with similar deck details were obtained from the Pennsylvania Turnpike Commission, the Allen County Highway Department in Indiana, the California Department of Transportation, and the Atcheson, Topeka and Santa Fe Railway Company.

Indiana

In 1967, Purdue University, in cooperation with the Indiana State Highway Commission, began laboratory and field studies to establish criteria for a precast full depth concrete deck system utilizing transverse prestressing.(1) Also included was an evaluation of joints, connections, and related items pertaining to the design, fabrication, and erection of prestressed panels. The implementation phase of the study consisted of the replacement of decks on two structures--Route 37 bridge over Bean Blossom Creek and Route 140 over Big Blue River. An interim report was published covering the construction phase of the project.(2)

The deck was replaced using precast elements on a bridge in Allen County, Indiana which carries Tonkel Road over Cedar Creek. This project was not a part of the Purdue study.

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(1) See Reference List

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Indiana State Route 37 Bridge over Bean Blossom Creek. This bridge is a single span steel pony truss structure constructed in the early 1900s with a cast-in-place reinforced concrete deck. The span is 127 ft. (38.7 m) and the roadway consists of two 14-ft. (4.3 m) cartways. The precast deck is 32 ft. (9.8 m) wide. Guardrail sections attached to the truss members are used in lieu of safety parapets. The original deck was replaced with a precast, prestressed concrete deck in 1970 (see Figure 1).

Figure 1. Roadway Elevation of Indiana Route 37 Bridge over Bean Blossom Creek

Originally the bridge carried Route 37 traffic through the area. Several years after the deck replacement project was completed, a new highway was constructed to provide a bypass of the City of Bloomington. As a result, the structure now carries local traffic only. Regular salting during the winter months is required.

Joint details used in the precast deck were based on laboratory work done in the initial phase of the Purdue study. These are male-female type joints. A thin neoprene sheet was provided in the joint to decrease stress concentrations and to assure a watertight seal between slabs. In addition,

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the joints were match-cast to assure a tight fit between the units. Except for the minor spalling of the joint edges due to poor match-casting, the joints are functioning well under repetitive loads.

Longitudinal post-tensioning was used which resulted in several problems in fabrication and erection. The need to accurately position the post-tensioning ducts was essential to their proper functioning. As with inserts, it was necessary to place and anchor the ducts in a manner which would prevent movement during the placement and consolidation of the concrete. During erection of the deck it was necessary to partially post-tension several slabs at a time in order to provide a good initial fit at all joints. The final post-tensioning was accomplished from one end and required removal of the approach pavement to provide sufficient room for the post-tensioning jacks.

Spring clips were used to hold the slab and support beams together. These proved to be costly to fabricate and did not function as intended. Bolts used to apply the clamping force to the clips were loosened before longitudinal post-tensioning to prevent loads being transferred into the support system. Figure 2 shows a detail of these clips.

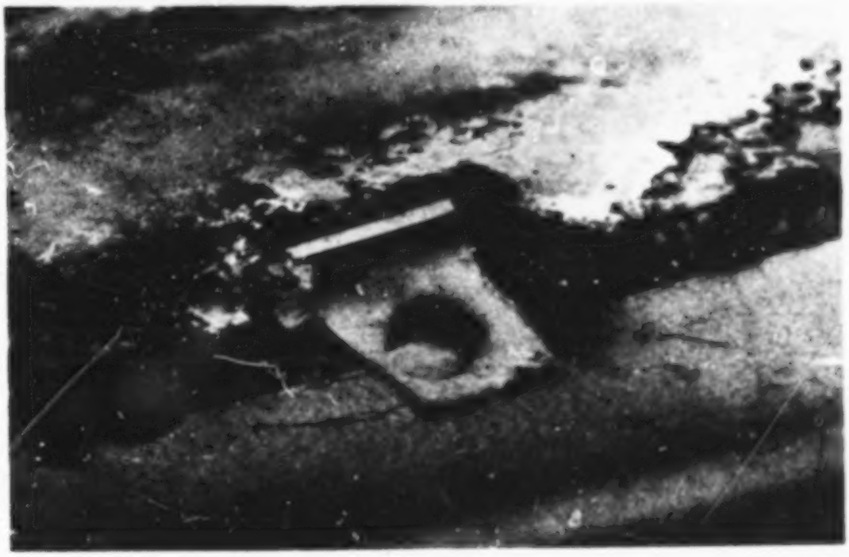

Figure 2. Typical Spring Clip Attachment Between Deck Slab and Support Girder.

Fabrication of the deck panels was not done efficiently since there was no indication that a continuing need would exist for special forms for casting the slabs. The contractor employed a system utilizing standard 4-ft. (1.2 m) casting beds rather than developing a set of specialized permanent forms. In addition, the inserts used to attach the spring clips to the slab were difficult to hold in the correct position during the casting of the slabs due to the form design employed by the contractor.

Erection of the precast slabs required some modification to be made to the floor framing. The fascia and first interior stringers on each side of the structure had to have plates welded to the top flange to attain the required elevation needed to match the profile and crown of the roadway. In addition, the stringers were not continuous over the floor beams, and another series of plates had to be welded to the floor beam top flange at each stringer location to provide a smooth bearing surface. Since a leveling bed was not provided, the relatively vertical position of the stringers had to be nearly perfect. Following placement of the deck slabs and completion of longitudinal post-tensioning, gaps between the supporting members and the slabs were filled with epoxy.

The entire erection process including post-tensioning required two days to complete.

Present structural condition of the bridge deck is good with only minor spalling at the transverse joints (see Figure 3). The underside of the deck is sound, with minor leaking at the transverse joints. The ends of the prestressing strands show little evidence of corrosion, and there are no transverse cracks which would provide evidence of possible corrosion of the prestressed strands (see Figure 4).

One major problem exists on the deck of this structure. The rubber expansion device on one quadrant of the bridge has failed and has been removed. The bolts holding it in position have sheared and exposed ends project into the cavity left by the loss of the joint material. The result of this condition is a constant impact on the slab and bridge support system as vehicles cross the bridge.

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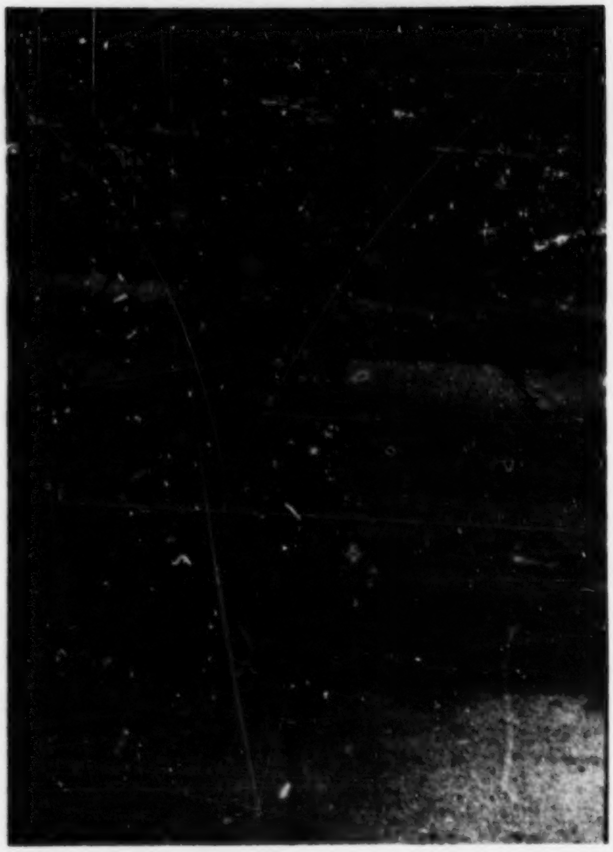

Figure 3. Typical Joint on Roadway Surface

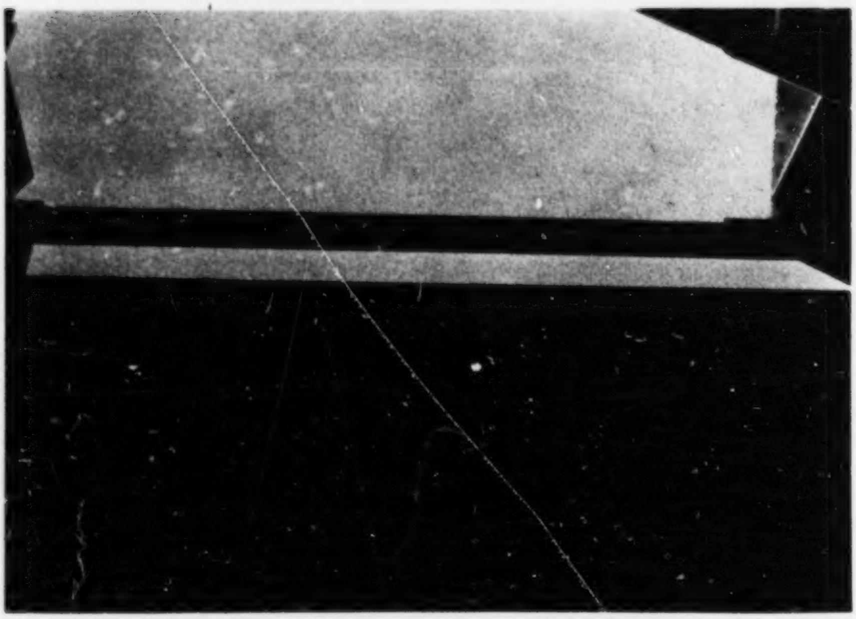

Figure 4. View of End of Slab Section at Curb

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Indiana Route 140 Bridge over Big Blue River. This structure, just south of Knightstown, is a three-span continuous steel bridge with one span at 70 ft. (21.4 m) and two at 60 ft. (18.3 m). The superstructure has been reconstructed and the beams placed at a constant elevation across the deck width. The roadway consists of two 12-ft. (3.7 m) cartways with 6-ft (1.8 m) shoulders, providing an overall width between curbs of 36 ft. (11 m). The curbs were cast-in-place after deck construction was completed. Roadway drainage is provided through blockouts in the curb.

The bridge was constructed in the early 1900s. The deck replacement project was completed in 1970 (see Figure 5).

Figure 5. Roadway Elevation of Bridge Carrying Indiana Route 140 over the Big Blue River.

The details of the precast concrete deck slabs are similar to those used on the Route 37 bridge already discussed, including the spring clips, joint details and longitudinal post-tensioning. The slab thickness, however, was not made constant as in the Route 37 bridge since the floor framing was constructed without cross slope. To develop the required crown, the slab thickness varies from 7-1/2 in. (191 mm) at the curb to 10-1/2 in. (267 mm) at the centerline.

Fabrication of the deck slabs was done in a manner similar to the Route 37 bridge. In order to eliminate concrete tension under all loading, strands were placed in a symmetrical pattern about the centerline of the slab. Since the slab was variable in thickness, the top row of prestressing steel was placed parallel to the top surface of the slab. To accomplish this, strand chairs made of reinforcing bars were necessary to hold the bars in the proper location in the slab. The chairs had to be strong enough to withstand the vertical component of the prestressing force applied to the strand.

Erection of the precast deck panels and completion of the post-tensioning operation required approximately two days.

The present condition of the deck is good. Several of the spring clips used to attach the slabs to the support beams are loose. There appears to have been no loss of post-tensioning force in the longitudinal direction and no evidence of concrete spalling except at the joints. The deck has no protective overlay and has required little or no maintenance. While traffic is light, there is occasional salting. The riding quality of the deck surface is good (see Figure 6).

Tonkel Road Bridge over Cedar Creek, Allen Co., Indiana. This structure, built by Allen County in the 1900s, is a 140-ft. (43 m) single span pony truss bridge constructed originally with a concrete cast-in-place deck. The deck was replaced in 1972 with a precast, prestressed modular deck system.

Several unique features were developed with the design of the deck replacement, including the method of attachment of the panels to the supports and the joint between adjacent slabs.

A fibrous expansion joint material was placed on top of the supporting beams prior to placing the slabs. The slabs were attached to the beam using

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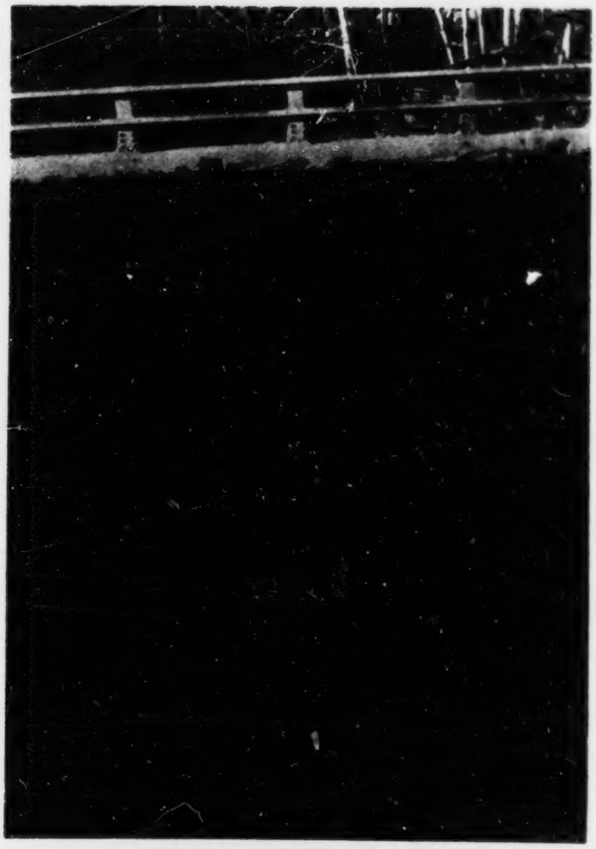

Figure 6. View of Roadway Surface - Route 140 Bridge

a "Z" clamp and bolt threaded in inserts placed during fabrication of the panel (see Figure 7).

In lieu of providing a shear resistant joint between slabs, a vertical faced butt joint with steel clips welded to steel plates embedded in the slab during fabrication was used. These clips were placed at several points along the transverse width (see Figure 8). This method of attachment necessitated the positioning of weld plate inserts in the deck slab prior to placing the concrete in the forms. The deck was then covered with an asphaltic concrete wearing surface which provided the required cross-slope on the bridge and also covered the slab connections. Longitudinal post-tensioning was used to provide compression in the longitudinal direction.

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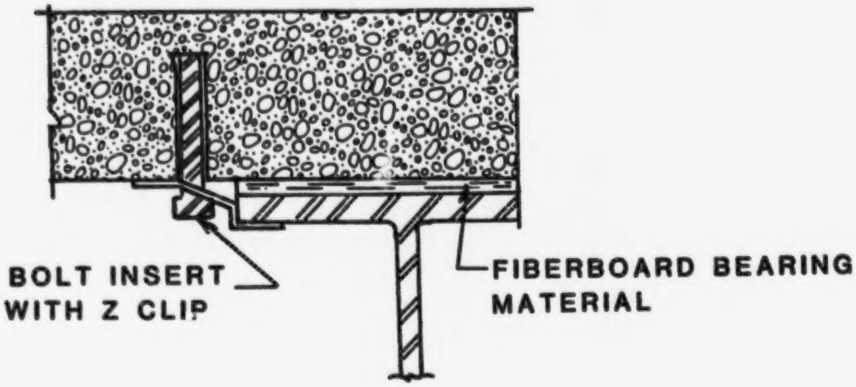
BOLT INSERT
WITH Z CLIP
FIBERBOARD BEARING
MATERIAL

Figure 7. Schematic of Fiberboard Bearing Material and Clamping Bolt. Tonkel Road Bridge.

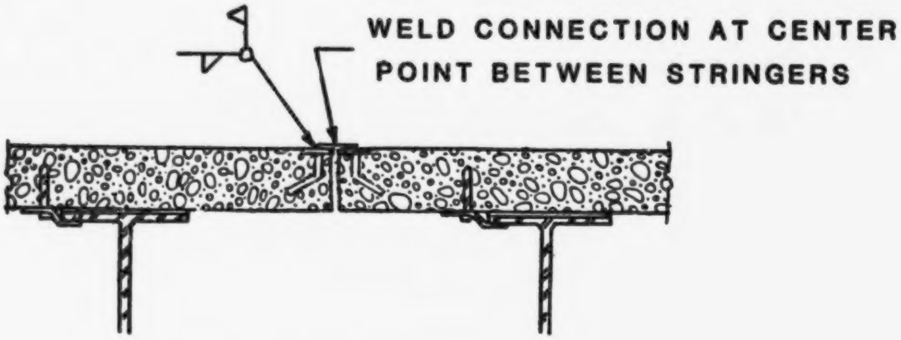
WELD CONNECTION AT CENTER
POINT BETWEEN STRINGERS

Figure 8. Schematic View of Weld Plate for Shear Resistance. Tonkel Road Bridge.

New York State Thruway Authority

The New York State Thruway Authority has used precast full-depth reinforced concrete deck slabs on several bridge rehabilitation projects. The Authority decided to develop this design using mild reinforcement instead of prestressing. They determined that in their area, the use of deicing salts could cause corrosion of the prestressing strands.

In lieu of special hold-down clamps, a positive connection between the slab and supporting beam was developed with a block-out and shear connection system. Several different systems were investigated by the Authority and its consultants prior to selecting the shear connection system finally used. Although the Authority does not design for composite action, this system will develop composite action as well as provide a positive connection to the support beams.

Slabs are cast with cut-out blocks positioned at predetermined locations over the beams. The slabs are then set on thick epoxy beds applied to the beams to provide uniform bearing. Shear connectors, normally stud type, are then welded to the support stringers and the cut-out areas are filled with an epoxy cement mortar.

Transverse joints are double female type filled with pourable epoxy mortar. Longitudinal post-tensioning is not used (see Figure 9).

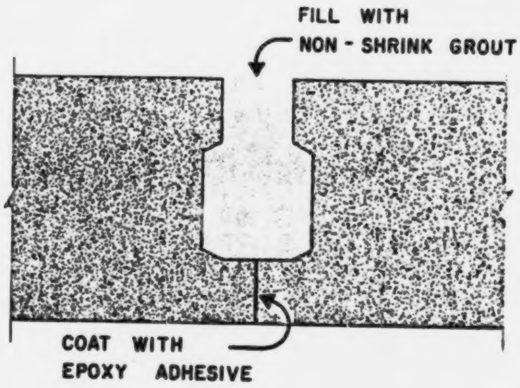

Figure 9, Double Female Type Joint

Tupical designs call for slabs to be cast in 4-ft. (1.2 m) widths. Panel lengths are sufficient to cover one-half of the bridge. A cast-in-place joint along the centerline of the bridge is used to join the two halves.

One bridge incorporating these details was inspected by the research team. This bridge carries the Thruway over Krumkill Creek. It was constructed with an impervious membrane protection system and was overlaid with a variable depth wearing surface. This bridge which has been in service for four years is in good condition, providing excellent riding quality. As reported by Thruway personnel, several other structures have been rehabilitated using this system and all are performing in a similar manner.

The Authority bridge engineering staff is well satisfied with this concept and believes that they have reduced the construction time for deck replacement projects and improved the quality of the deck with no significant change in construction cost.

Pennsylvania Turnpike Commission

In 1979 a deck replacement contract was awarded for the Clark Summit Bridge located on the northeast extension of the Pennsylvania Turnpike. This 1627-ft. (496 m) high level structure employs a twin girder steel framing system with floor beams and cantilever brackets supporting stringers which in turn support the reinforced concrete deck slabs. Spans are continuous with intermediate hinges. Construction took place in 1957. The southbound bridge deck was replaced in 1979 and the northbound is scheduled for replacement in 1980.

The replacement deck is designed as a full-depth precast system using mild reinforcement. Panels are 4 ft. (1.2 m) in width and 30 ft.± (9.3 m±) in length. Design is based on HS 20-44 truck loadings per AASHTO specifications.(3)

The geometrics of the framing system require that provision be made for variations in elevation between the roadway surface and the supporting members. This difference varies by 6 in. (152 mm) in certain areas. The plans call for the removal of the deck, followed by a field survey of the steel framing system to establish existing elevations. The tops of these members are cleaned by sandblasting. Neoprene strips are then glued to the edges of the stringers to create a suitable side form for the placement of epoxy grout between the bottom of the precast slab and the top of the supporting element.

The flange plates on the main girders vary in thickness, creating a series of sudden changes of dimension between the bottom of slab and top of steel. This variation in dimension has been taken care of by providing a haunch in the precast slab of sufficient depth to nearly fill the void. For this reason, the slabs cannot be fabricated until the survey has been made, and the required haunch depth at each slab position field determined. This also means that slabs are not interchangeable, but have to be produced and delivered in a precise sequence. To prevent delays in the event a unit is damaged, or otherwise found unacceptable, two slab sections for which no haunch has been provided are kept on the site. Additional form work will be required to provide the required haunch for these substitute slabs if they are used.

The transverse joint is a double female type similar to that being used by the New York Thruway Authority already described. The gaps at the top of the slab are 2 in. (51 mm) to permit use of a non-shrink grout instead of the pourable epoxy specified for the New York Thruway bridges.

Four longitudinal post-tensioning type ducts are cast into the slab. These are used to carry threaded rods through the slab. The purpose of the rods is not to post-tension the structural deck, but to pull the individual units as close together as possible during placement. The threaded rods have sleeve nuts attached to one end for this purpose. The slab is positioned, and the rods are threaded into the exposed sleeve between slabs and tightened. The threaded sleeve on the far side acts as a nut to pull the slabs together and also acts as the insert for the next rod.

The deck will have a 1-1/4 in. (32 mm) layer of latex modified mortar applied for protection following completion of the deck replacement. The top layers of reinforcing bars are epoxy coated for additional protection against corrosion.

The contract schedule specified that six panel units be placed each working day and the contractor developed his production accordingly. Although the number appeared to be a reasonable goal to achieve during a normal eight hour day, experience showed that the goal was achieved routinely in a four-hour work period.

The initial contract for redecking one-half of the bridge was completed last fall while maintaining traffic on the other half of the structure. A contract will be awarded for the remainder of the work in 1980, at which time modifications in production rate requirements will be made.

California

A deck replacement was completed on the High Street Overhead Bridge located near Oakland, California.

The original deck was saw-cut and removed in 60- to 80-ft. (18 to 24 m) sections, and precast slabs 12 ft. (3.7 m) wide placed in the voids. The slabs were made composite with the support beams by attaching shear studs through cutouts provided in the precast slabs. Mild reinforcement was used throughout.

To achieve the desired elevation of the slab several two-headed bolts were placed through inserts in the slab. These bolts extend one to two inches below the slab and can be adjusted to provide the required elevation. While the slabs were held in this way, wooden side forms were attached to the stringer top flange to create a form for the placement of fast-setting concrete between the slab and steel girder. This same material was also used to fill the cutouts after placement of the stud shear connectors.

Atcheson, Topeka and Santa Fe Railway Company

The Atcheson, Topeka and Santa Fe Railway has developed precast, prestressed deck slab construction standards for use in replacement of timber decks. The concept differs from those already discussed in that there are no mechanical attachments between the slabs and the supporting structure. Bolts with spring clips are used at two points on each slab to temporarily attach the slab to the supporting girder during construction.

The procedure specifies that the girder flanges are to be sandblasted following removal of the timber deck. Plywood forms are attached to the edges of the flange and a pourable epoxy bedding material placed. Slabs are then erected while the mortar is still plastic. The spring clips are used to hold the slab in place while curing of the mortar is completed.

Joints between panels are plane-faced epoxy coated. No other shear transfer device is used.

The railroad reports that the epoxy mortar between the slab and beam develops sufficient shear capacity to insure composite action between the girder and slab.(4) This provides sufficient increase in live load capacity of the structure to overcome the additional dead load resulting from the concrete deck. It also eliminates the need in many instances to make extensive structural repairs to the supporting girders to increase capacity.

There are differences between highway and railroad loadings which would prevent the development of composite action through bond. One of these is the probability of uplift on highway bridge decks that could weaken this bond due to location of loads on the deck which can be accurately determined for railroads with fixed wheel location, but not for highways. The procedure is, however, one that should be expanded and tested for possible use on highway bridges.

CHAPTER III. DESIGN CONSIDERATIONS

DESIGN CRITERIA

Criteria for the design of bridge decks has been established by AASHTO and is revised periodically when there is an agreement among the states that a change is needed. The criteria is developed primarily for the routine type of construction where the slab is continuous over a series of supports (stringers or girders). The prestressing or post-tensioning concept can be developed utilizing most of the same criteria although certain areas should be modified.

Allowable Tension

Permitting some tension to exist in prestressed slabs may result in a reduction in prestressing force required. The potential adverse effect is that tension cracks can occur at the surface of the concrete and if this surface is one that is treated for snow and ice control, contaminants can permeate the slab at an accelerated rate. In the case of salting, this could cause deterioration of the slab at a faster rate.

If the prestressing strands are eccentrically located, tension can be permitted on the underside of the slab in the area of positive movement, but not permitted on the top or roadway surface in the area of negative bending. In those areas of the country where the underside of the slab rather than the roadway surface is exposed to salt spray, this could be reversed, permitting tension in the negative bending area and not in the positive areas.

Whenever tension is permitted in a precompressed area, mild reinforcement must be provided to take the tension force. This is required by the AASHTO specifications.

Strand Placement

Strand patterns can be placed in a single or double layer parallel to the surface of the panel or can be draped to follow the line of maximum stress. The latter can reduce the amount of prestressing required when the loads are primarily fixed and the maximum stress line remains relatively constant. In slab design, continuous over several supports and spans in the 6-ft. (1.8 m) to 10-ft. (3 m) range, live load is the predominant factor and therefore the line of maximum stress varies significantly. Furthermore, the magnitude of stress and the direction (tension-compression) vary. As a result, draping strands is not practical.

Distribution Steel

Current AASHTO specifications call for distribution steel to be placed in the bottom layer of reinforcing for cast-in-place slabs. Distribution steel is also specified for precast prestressed concrete form panel construction. Section 1.6.26 states that 0.11 in.^{2} per ft. (233 mm^2 per m) of bridge width must be placed in the precast panel in the longitudinal direction of the bridge. It also specifies 0.25 in.^{2} per ft. (529 mm^2 per m) of bridge width for temperature and shrinkage in the cast-in-place portion of the deck with the top steel.

In research performed at the Pennsylvania State University (5) and by the Texas Transportation Institute(6) on prestressed concrete bridge elements, the requirements of the AASHTO specifications for distribution steel were supported. In a study prepared by Purdue University (1) there was no distribution steel provided. Longitudinal post-tensioning was provided and the deck has performed in a satisfactory manner.

The conclusion of the research team on this study is that the same distribution steel should be used for full-depth precast slabs as required for cast-in-place slabs until refinements if required are substantiated by additional studies. The current AASHTO requirements should be considered a minimum for either prestressed full depth slabs or precast slabs with mild reinforcement. Top steel should have a minimum cover of 2 in. (51 mm) and bottom steel 1 in. (25 mm).

Variable Depth Slab

In the design of longitudinal beams and girders, it is often desirable to provide a variable depth member. This can result in a more efficient use of materials and a more economical design.

Concrete deck slab design based on current specifications, requires the same moment capacity for positive and negative bending. Cantilever moments can require greater capacity. Therefore, there is no advantage in a variable depth slab with the possible exception of the cantilever portion. This area should be investigated for each design.

Longitudinal Post-tensioning

Longitudinal post-tensioning provides several benefits when used in conjunction with precast concrete slabs. The shear resistance between adjacent slabs is increased through friction and can provide sufficient resistance to eliminate the need for shear keys. Post-tensioning will also provide a tighter joint between slabs and therefore improve watertightness.

A third benefit is the development of a two-way slab system through longitudinal post-tensioning and transverse prestressing. This can reduce the incidence of tension cracks in the slab, improving the resistance to penetration by moisture and improve performance of the slab under freeze-thaw cycles.

Based on these considerations, post-tensioning is recommended. Unless required by structural computations to be greater, post-tensioning a minimum tensioning force to produce 40 psi (0.28 MPa) should be used. This figure is based on the results of the Purdue study.(1)

Design Specifications

Except under unusual circumstances, the existing formulas provided by the AASHTO Standard Specifications for Highway Bridges should be used for most deck design applications. When the girder spacing increases to such an extent that the standard formulas become too conservative, a rigorous design should be employed. Any analysis procedure which adequately accounts for the type support as well as load location variations is adequate. For deck slabs generally used on today's overpass type structures, the savings provided by the more rigorous design method may prove too small to be worth the additional time required for a rigorous analysis.

DESIGN DETAILS

There are a number of details that must be carefully developed with a precast deck system. These include joints, slab bearings on support members, geometrics, post-tensioning ducts and protective systems.

Joints

Proper joint designs must be developed to carry applied shear and/or moment. The joint must also be waterproof and provide a smooth and safe riding surface.

Transverse. Several types of joints for use between precast panels were reviewed. The simplest type is the butt joint which depends upon either frictional resistance developed through longitudinaly post-tensioning to transmit shear, a bonding agent, or mechanical devices embedded in the panels. Alignment of the panels is difficult but an overlay wearing surface can improve riding characteristics. The butt joint has been used effectively on the Santa Fe Railway bridges and the Tonkel Road bridge in Indiana.

A second joint type is the double female type joint, used by the New York State Thruway Authority, and recently by the Pennsylvania Turnpike Commission. This joint is made to form an open pocket when two slabs are placed together (see Figure 9). The center portion has the widest opening, narrowing down somewhat at the top, and providing a closed or nearly closed joint at the bottom.

When filled with a suitable grout, the result is a shear type double keyed joint. The opening at the top of the slab allows for the proper placement of the material within the cavity. This joint type has proven to be of good overall quality and to provide dependable shear resistance.

The third type of transverse joint is a male-female or keyed joint. This joint depends upon a close fit at all locations for its strength. Several configurations for this type of joint were investigated as part of the Purdue Study (see Figure 10).

Of the three configurations, the flat type proved to be the simplest and most effective. This joint was used on the two bridges built by the Indiana State Highway Commission in 1970.

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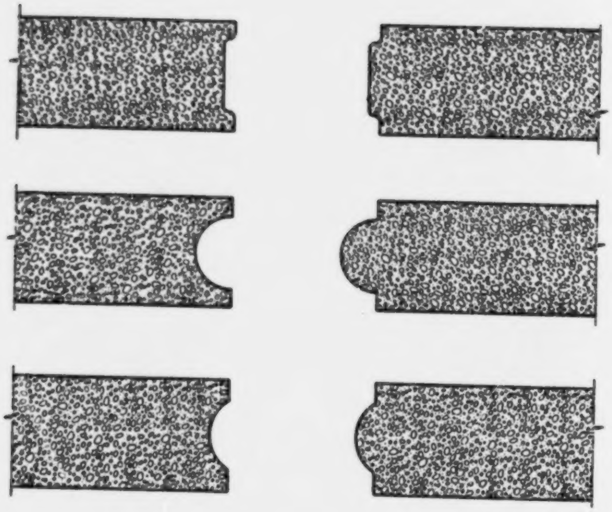

Figure 10. Commonly Used Shear Key Configurations

Longitudinal. Where multiple width slabs are used, a longitudinal joint is required. This could be an open type or a load transfer joint for shear and for moment, depending on the location of the joint in the slab.

Where continuity is required, a cast-in-place concrete section is used to connect adjacent precast slabs. Mild reinforcement is provided which extends from the precast slab a sufficient distance to develop anchorage. Cast-in-place concrete is added to complete the joint.

Depending on the location of the joint in relation.to the support system, several alternate possibilities were reviewed. If the joint occurs over a support beam and it is not necessary to provide continuity, a lap type joint can be provided. This joint provides shear resistance through the overlap of the concrete sections and through the use of shear connectors which extend through both slabs to bond the two units together. An epoxy material applied to the two surfaces at the joint provides additional bond capacity. (see Figure 11)

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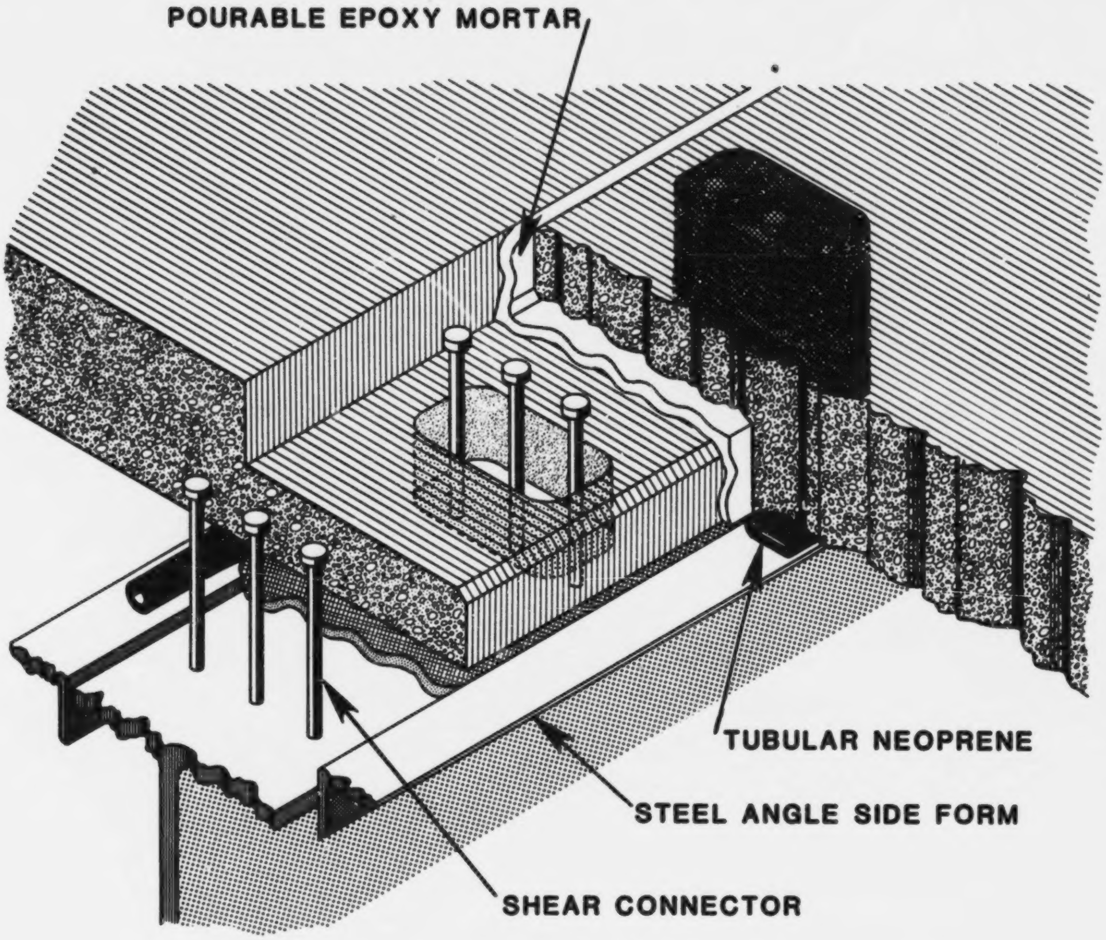

Figure 11. Typical Lap Type Longitudinal Joint

A similar joint configuration is possible where the joint occurs between support beams. When used it is necessary to treat the span between girders as two separate cantilevers. Shear resistance guarantees that both slabs deflect equally and will distribute the load over the two cantilevers. The cantilever moment will normally be greater than the interior slab positive design. Provisions must be made to allow for this increase. Keyed joints discussed under transverse joints could also be used in this situation.

In cases where neither shear nor moment transfer is required, an open joint can be used. Details should include sealing to prevent surface run-off from passing through the joint.

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When an open joint is used, care must be taken in the design of the cantilever portion of the slab to insure that sufficient prestressing has been developed before reaching the point of maximum moment. Figure 12 illustrates this graphically. Where the distance between the joint and the first support is less than the required development length of the strand, the moment capacity may not be equal to that developed through applied loads.

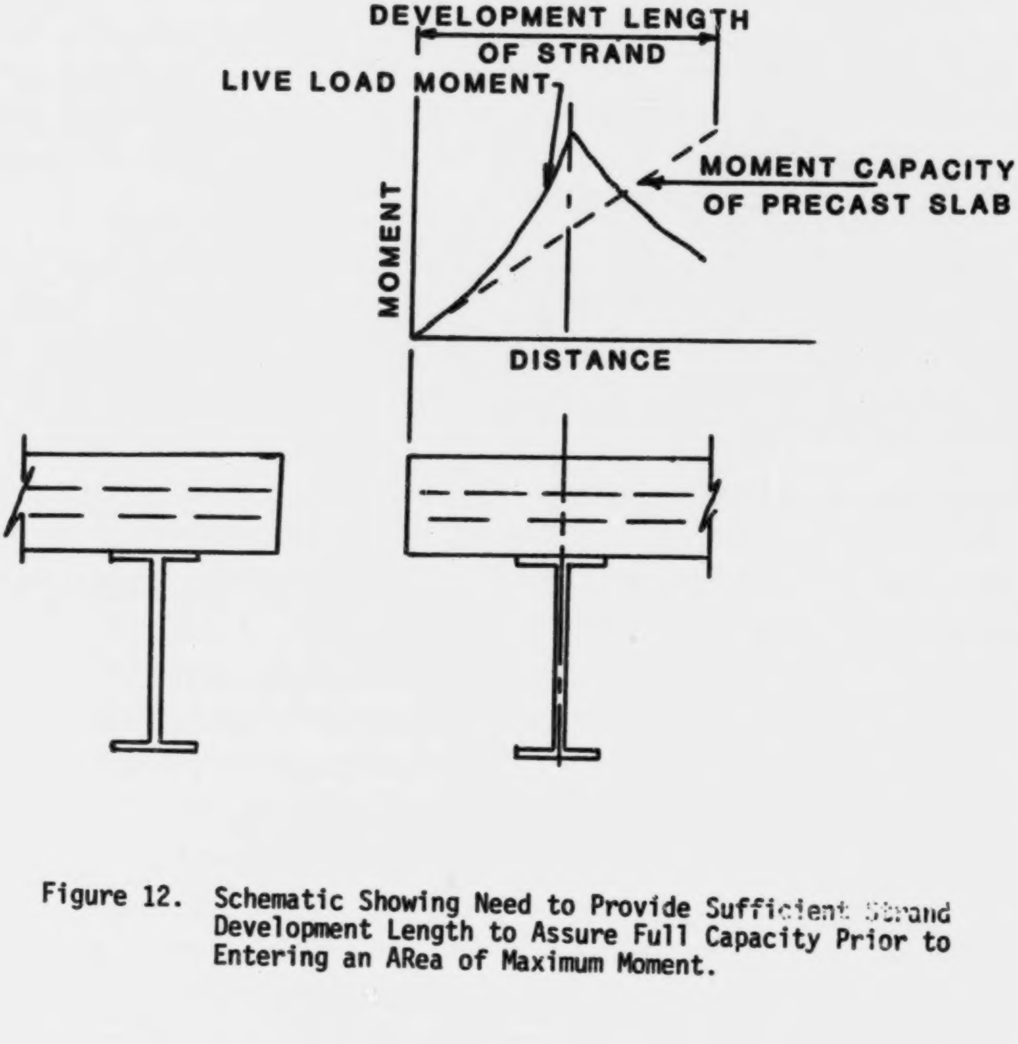
DEVELOPMENT LENGTH OF STRAND
LIVE LOAD MOMENT
MOMENT CAPACITY OF PRECAST SLAB
MOMENT
DISTANCE

Figure 12. Schematic Showing Need to Provide Sufficient Strand Development Length to Assure Full Capacity Prior to Entering an ARea of Maximum Moment.

This could be overcome by:

(a) Increasing the prestress force,
(b) Adding mild reinforcing steel,
(c) Coating the strands after stressing with an epoxy adhesive to increase bond between the concrete and the strands.

Item (c) is a new concept and should be thoroughly tested prior to use.

Filler Material. The filler material used at the joints should provide a waterproof seal and in certain types of joints, provide bond between the adjacent slabs. Where shear transfer is required, epoxies or cement grout are usually specified. In those instances where sealing is the only concern, neoprene or rubber joint material can be used.

Pourable epoxy material with a relatively long pot-life is a material which can be spread or brushed onto the edge of a precast slab prior to placement and remain workable long enough to permit the adjacent slab to be placed against the joint. The epoxy bonds well with the concrete surfaces.

Epoxies are now readily available with variable pot-lives to comply better with the needs of the application. For example, an epoxy with a fairly short pot-life is used to fill the space between two slabs with a female-female joint configuration. Epoxy placed on a male-female or butt joint or at the base of a female-female joint prior to erect-on of the adjacent slab should have a longer pot-life to permit placement and adjustment of the new slab before hardening. Therefore, the application requirements and the strength requirements will dictate the characteristics of the epoxy.

Due to the high cost of epoxies it could be more economical to use a non-shrink grout as a substitute for epoxy where possible. This can be used on the female-female type joint as was done on the Clark's Summit bridge in Pennsylvania.

Where the joint material must extend to the surface of the slab, skid resistance of the material becomes important. Epoxy resins have very poor skid resistance and must be accompanied by special surface treatment to improve skid resistance. Cement grout has better skid resistant qualities and is compatible with the existing deck texture and color.

Connection to Support Structure

Except when composite action is required, the usual attachment between the slab and the supporting structure for cast-in-place bridge decks has been the natural bond developed between the two units. The girder flange is usually keyed to the slab to provide lateral stability for the top flange of the supporting member. In composite design, mechanical shear connectors are provided.

In precast deck construction, details must include provision for a positive attachment to the supporting structure. Several devices have been used including bolt clips with threaded inserts, channel connectors, and stud type shear connectors. These have already been described in Chapter II for use with steel support members.

When modular deck units are used with concrete beam supports, different anchorage details are needed. One example of a suitable detail to accomplish anchorage in new construction is shown in Figure 13. A flat plate is anchored to the concrete flange by studs welded to the plate. The plate can be fabricated flush with the flange surface or positioned on top of the flange surface. The top of the plate serves as the base for the attachment of the welded studs used to anchor the deck to the beam. If additional shear capacity is needed, the plate can have slots cut into it to provide a cavity which, when filled with epoxy or non-shrink grout after placement of the slab, provides a series of shear keys. An alternative is to weld bars to the plate at predetermined intervals which will increase the shear capacity of the connection.

A similar detail can be used with existing prestressed concrete beams. The existing beam usually has reinforcing steel stirrups that are embedded in the cast-in-place slab (see Figure 14).

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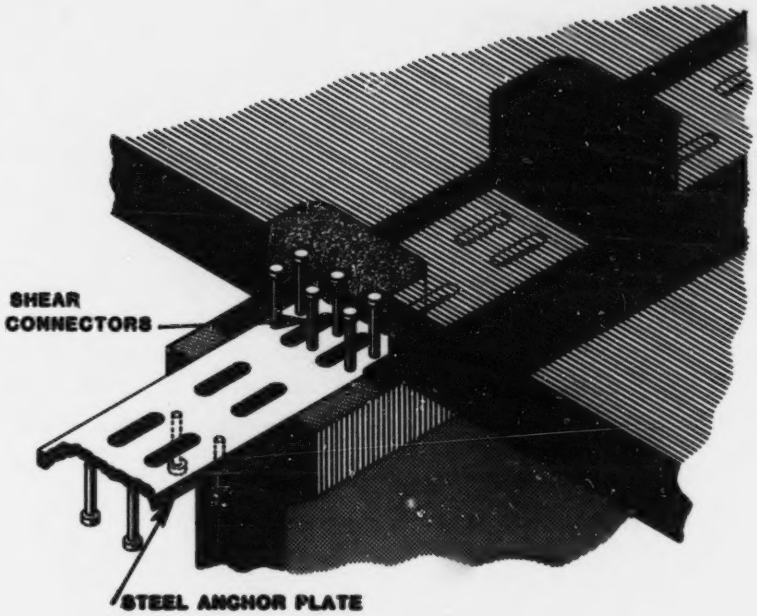
SHEAR
CONNECTORS
STEEL ANCHOR PLATE

Figure 13. Steel Plate Embedded in Precast Girder

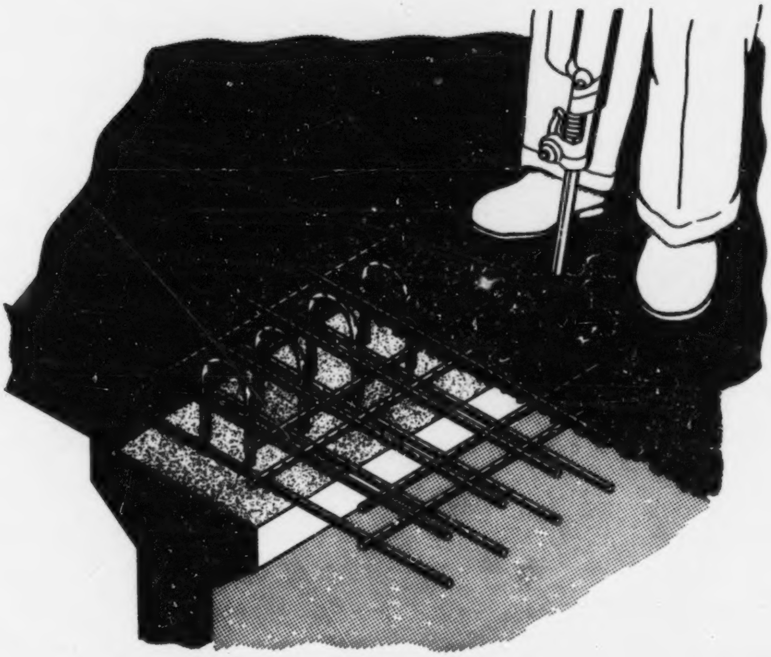

Figure 14. Prestressed Concrete Beam Stirrup Arrangement

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The concrete is removed from the top of the beam by jackhammers exposing the reinforcing steel. The deck reinforcing steel is removed and the stirrup bars are cut. A steel plate is placed on top of the beams. The bars are then bent down to contact the top of the plate and welded in position (see Figure 15).

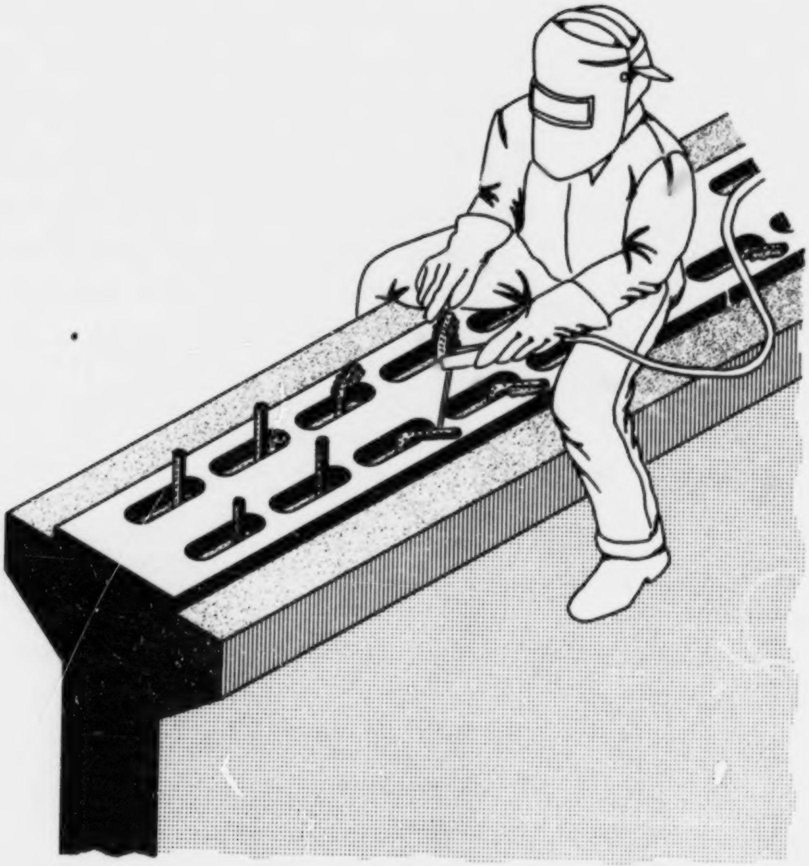

Figure 15. Shear Plate Attached to Concrete Beam.

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The final step occurs after the placement of the slab. The studs are welded to the plate through the slab blockout provided and the bearing material is placed (see Figure 16).

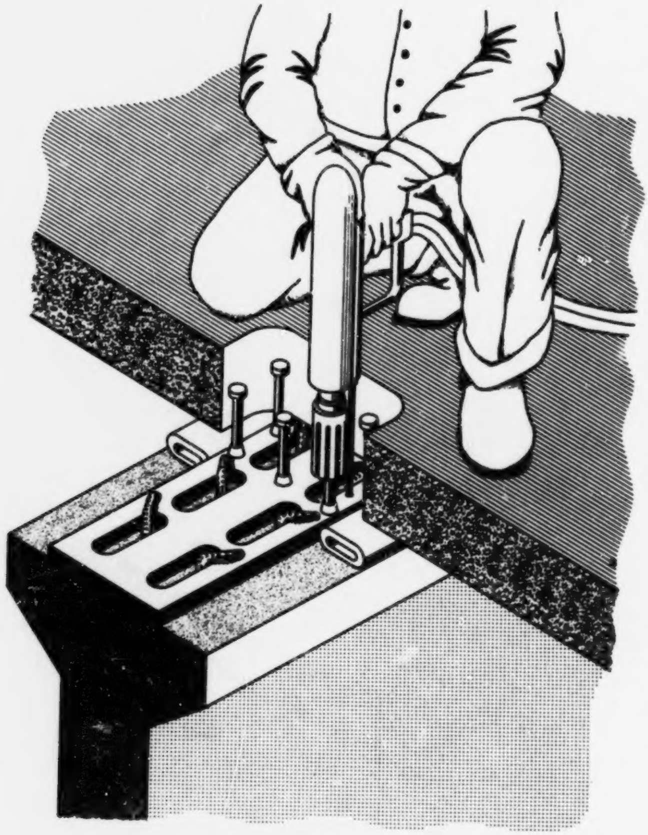

Figure 16. Stud Welding to Shear Plate.

High strength bolts can be used to connect a precast slab to the steel support structure. Holes are drilled in the top flange at locations conforming with preformed openings in the deck slab. The openings in the concrete serve as a template for field drilling the holes in the beam

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flange. The surface of the slab adjacent to the preformed hole is recessed to permit the bolt head and washer to be flush with the top surface of the deck. The bolts are inserted before placement of the bearing material but not tightened until the bearing material is placed and has achieved its full strength. Tightening the high strength bolts develops compression between the slab and beam, thereby providing horizontal shear resistance through friction.

On prestressed concrete beams a similar system can be used. In this instance, the shear device is a threaded high strength steel rod with the threaded end projecting into the slab void. A plate insert is provided which permits tightening of the nuts against a steel bearing plate. The bolts are tightened after the bearing material has reached its full strength (see Figure 17).

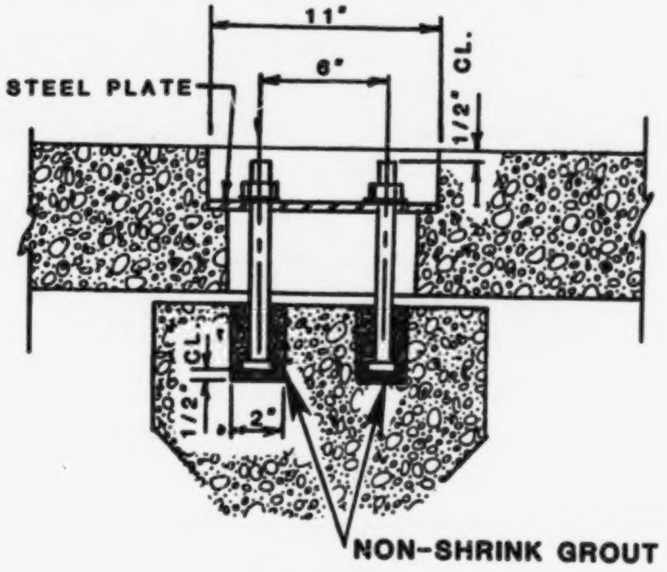

Figure 17. Precast Slab Bolted to Concrete Beam

Horizontal Shear Connections

It is often a requirement to develop composite action in a replacement deck in order to preserve the design structural capacity of the bridge or to increase the capacity of a deficient bridge. Several of the procedures already discussed can be used to develop the required horizontal shear between the deck and supporting element. It is possible to develop bond strength through materials used as bearing material which can provide the shear resistance necessary to provide lateral support for the top flange. The development of bond between the slab and support system through the support material can also serve to assist in the development of a reliable longitudinal shear resistance necessary for composite action between the slab and support girder.

The modular precast slab system developed by the Atcheson, Topeka and Santa Fe Railway has no permanent mechanical connection between the slab and the supporting girder. Spring clips are used to provide a temporary connection while the epoxy mortar placed between the girder and slab hardens to develop sufficient bond. Tests have verified that the slab and beam act compositely under live load. There is no conclusive research on test results for highway application that substantiate the development of adequate bond for composite action using this procedure. There is sufficient evidence to suggest that bond capacity can increase the horizontal shear strength of the overall connection to the point where relatively few mechanical connectors are required. In such a case, the precast slab could provide equivalent composite section capacity without the large section losses needed to accommodate the shear transfer system. Such a consideration can add an important benefit to the concept of precast deck units.

Geometrics

Today's bridges carry geometrically complex roadways. Horizontal curvatures with variable rates of superelevation concurrent with vertical curvature are commonplace. Further complications are introduced when the roadway width must be varied or the horizontal geometry requires spirals. Deck replacement systems and new bridge construction must incorporate details to accommodate this geometry. In addition, the system must be designed to accommodate skewed bridge supports.

Skew. To accommodate bridge skew it is necessary to place the modular deck components parallel to the skew angle, requiring longer units. It is also necessary to skew the leader form for casting each slab section. In addition, if longitudinal post-tensioning is used, the effect of that force and its vector components must be taken into consideration, making analysis more complex. On decks with excessive skews, longitudinal post-tensioning should be eliminated and other procedures employed to achieve load transfer or uniform action between slabs.

Vertical Alignment. Superelevation, transition areas, and vertical curves can be handled in a satisfactory manner in the precasting operation, but due to irregularities in the supporting elements, create field problems. In those situations, a system that includes provision for field adjustments in vertical alignment is mandatory.

Several systems have been used in the past. One involves the placement of a non-shrink or epoxy grout on the top flange of the support members to an elevation higher than the desired elevation. The slab is placed and the excess material squeezed out at both sides of the slab. This method provides no positive elevation control during erection since there is no mechanism available to prevent more material from being displaced than desired. As a result, the slab must be raised and shimmed if the initial placement is below the desired elevation. This could result in the loss of adhesion bond between the slab and the support girder. Whenever the bedding material is placed prior to the slab, similar problems can occur.

Another method employs steel or neoprene pads fabricated in various thicknesses and used as shims under the slab. These are placed between adjacent slabs permitting one plate to support adjacent slabs simultaneously and also provide the desired vertical alignment (see Figure 18).

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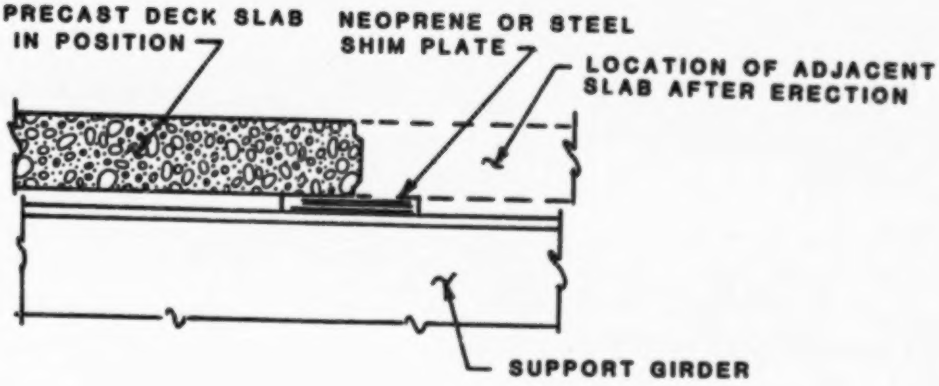
PRECAST DECK SLAB IN POSITION
NEOPRENE OR STEEL SHIM PLATE
LOCATION OF ADJACENT SLAB AFTER ERECTION
SUPPORT GIRDER

Figure 18. Shim Pads Under Slab

On steel beam or girder structures, structural angles can be welded to the top flange to support the precast slabs. This system provides a constant longitudinal support for the deck slab. The supports provide a side dam for the placement of the bedding material after the slab has been placed. This system also effectively eliminates the problem of variable flange thicknesses often encountered on plate girder bridges (see Figure 19).

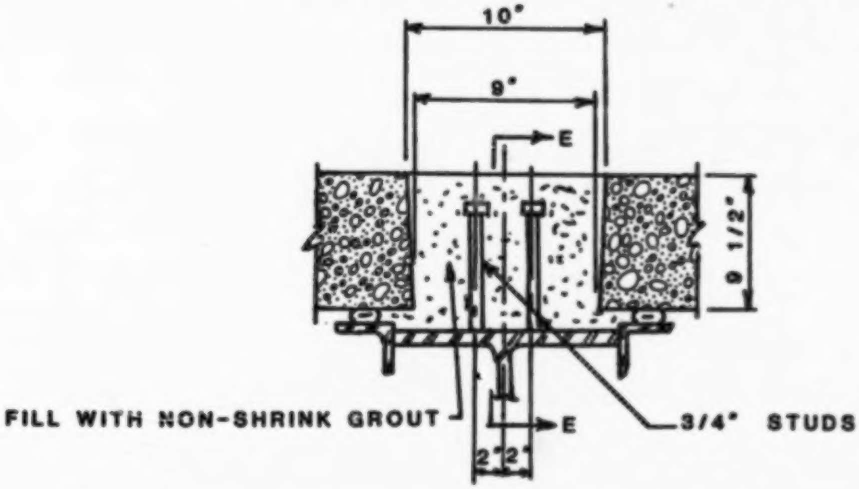
10"
9"
E
9 1/2"
FILL WITH NON-SHRINK GROUT
E
3/4" STUDS
2"
2"

Figure 19. Steel Angle Support System

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This system can be modified for use on new concrete girder construction. Inserts embedded in the concrete provide surfaces on which to weld the support angles.

A concept developed during the study that is considered to be the most versatile provides a means of vertical adjustment and support during erection. It consists of a long bolt, extended through a threaded insert in the slab. The support system provides the necessary bearing surface, and the slab can be adjusted to the proper elevation by increasing or decreasing the extension of the bolt end below the bottom of the slab. The number of bolts equired is dependent on the dead load of the slab and the needs for vertical adjustment. After the work is completed, the bolt can be removed or cut off below the concrete surface. The void is filled with a mortar or epoxy material (see Figure 20).

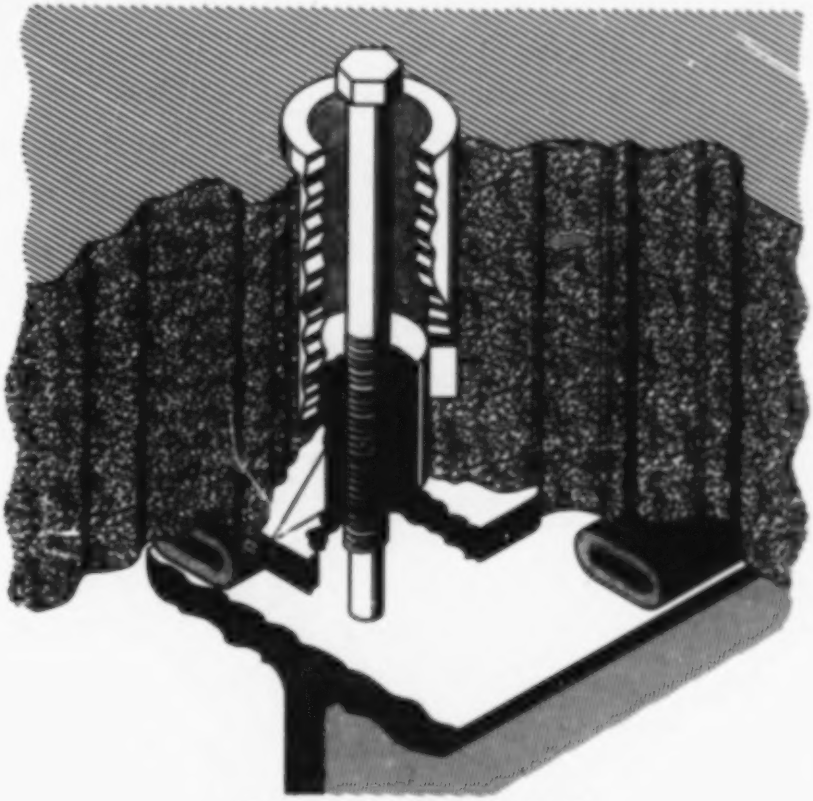

Figure 20. Bolt Support

Horizontal Curvature. Curved roadways create a situation which could require slabs to be fabricated with sides in a radial configuration. Providing the structure has radial piers and abutments, this procedure will provide adequate results. However, fabrication cost will increase significantly since the form work must be custom made for each structure.

Deck Protective Systems

One of the major bridge maintenance problems is the rapid deterioration of cast-in-place bridge decks. This is particularly evident in those regions of the country where snow and ice control chemicals are used or where the deck is exposed to salt spray or similar harsh environments.

A number of protective systems have been developed and implemented with varying degrees of success. These include those that attempt to seal the surface from impregnation by harmful chemicals (e.g., membrane or overlay systems), those that provide a protective coating to the steel reinforcing (e.g., epoxy-coated re-bars), and those that provide a physical repellant to harful ions (e.g., cathodic protective system). At the present time, the most affective systems are rigid overlays and/or epoxy-coated re-bars.

Since the same factors that have caused the rapid deterioration of cast-in-place bridge decks are present with precast modular deck systems, a protective system should be provided. Dense concrete or latex modified concrete overlays with epoxy coating for mild reinforcing will provide the most effective protection available at the present time for modular systems also.

Prestressed concrete offers many benefits which could affect the impermeability of the final product and improve its resistance to harmful penetration. Prestressing eliminates open cracks in the surface of the concrete perpendicular to the direction of the prestressing force. In addition, the prevention of cracking due to concrete tension under live load prevents the development of deep cracks which can permit the intrusion of clorides directly to the area of the reinforcing steel. Tests have shown, however, that prestressed concrete remains porous and therefore a deck protection system should be specified.

CHAPTER IV - DESIGN CONCEPTS

GENERAL DISCUSSION

Example design concepts were developed to demonstrate through specific application the ideas brought out during the course of the research project. A number of useful ideas were found during the state-of-the-art phase of the project and during the on-site investigations. Other ideas were developed by the research team. A number of these are included in the design concepts.

Each design example illustrates a technique that has been used successfully or provides a new technique for utilizing precast prestressed deck modules to their greatest advantage.

Design Concept Number 1 provides the additional deck slab width by increasing the deck overhang, without increasing the deck thickness. It also illustrates the application of deck modules on skewed supports, utilizing incremental longitudinal post-tensioning. A lap joint is used at the center of the bridge to permit half width rehabilitation.

Design Concept Number 2 demonstrates the use of longitudinally prestressed panels adaptable to girder floor beam type bridge structures. It includes the use of leveling bolts to achieve required deck elevation without the need for rigid forms.

Design Concept Number 3 presents a typical precast prestressed concrete superstructure support system, with typical details for making attachments to the deck slab to provide composite action for superimposed dead loads and live loads.

Design Concept Number 4 provides a comparison of the increased efficiency of prestressed deck elements by using a support spacing much greater than would generally be considered economical.

The ideas provided through the four concepts cover a wide range of possible application. The first two are for deck rehabilitation and the last two for new bridge deck construction. These ideas can be used interchangeably to solve a number of common bridge deck construction problems.

DESIGN CONCEPT NUMBER 1 - BRIDGE DECK REPLACEMENT

Design Concept No. 1 illustrates the applicability of transversely prestressed precast bridge deck elements on an existing structural support configuration. The concept also shows the value of increasing the cantilever dimensions of a bridge deck to increase usable roadway width.

Critical areas of concern are:

1. Traffic Maintenance. The structure is serving a large population area, closing the bridge would be a major inconvenience to the traveling public. This study assumes that only half the bridge can be closed to traffic at any time.

   The traffic on the cross street beneath the structure, while less important, must be maintained without interruption.

2. Speed of Erection. Two 10-ft. (3 m) traffic lanes can be provided on one-half of the bridge which will create traffic hazards and tieups throughout the reconstruction period. It is essential that the work be completed in the shortest possible time. Therefore, timing and coordinating the various erection procedures are essential.

Existing Structure

The existing structure has a simple span steel multi-beam bridge superstructure with an overall length of 78 ft. (24 m), over a two-lane crossroad. The structure is designed for non-composite action. The substructure units are skewed at an angle of 12^0 from the perpendicular. The original structural concrete deck slab is 7 in. (178 mm) thick.

The existing roadway is 52'-6" (16 m) wide, including four 12-ft. (4 m) traffic lanes with a 2'-3" (.7 m) shoulder area. New Jersey type safety parapets 2'-0" (.6 m) wide complete the deck for a total width of 56'-6" (17 m) from out-to-out of safety parapet. There are nine steel wide flange support beams spaced 6'-6" (2 m) on center. The bridge deck surface incorporates a 3/16"/ft. (16 mm/m) cross-slope downward on each side of the centerline. For a typical cross-section of the existing structure see Figure 21.

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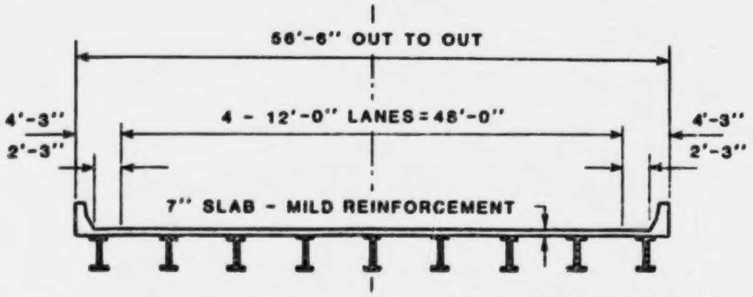
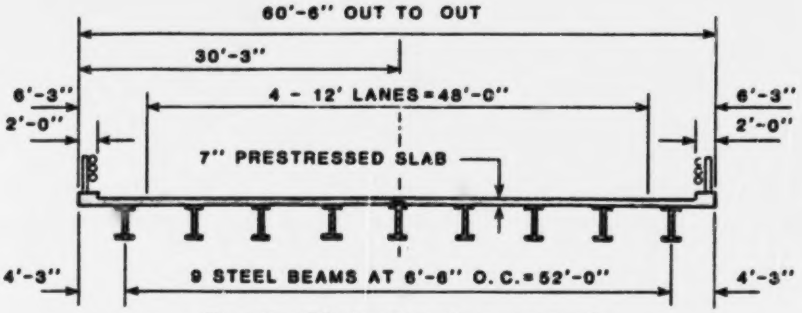
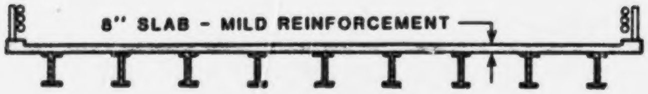

56'-6" OUT TO OUT
4'-3"
2'-3"
4 - 12'-0" LANES=48'-0"
4'-3"
2'-3"
7" SLAB - MILD REINFORCEMENT

Typical Cross-section of Existing Bridge


60'-6" OUT TO OUT
30'-3"
6'-3"
2'-0"
4 - 12' LANES=48'-0"
6'-3"
2'-0"
7" PRESTRESSED SLAB
4'-3"
9 STEEL BEAMS AT 6'-6" O.C.=52'-0"
4'-3"

Deck Slab Replacement
Transversely Prestressed Precast Slab


8" SLAB - MILD REINFORCEMENT

Deck Slab Replacement
C.I.P. Slab - Mild Reinforcement

Figure 21. Typical Cross-Section
Design Concept Number 1

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Proposed Deck Replacement

The proposed bridge deck is increased in width to 60'-6" (18 m) by increasing the overhang dimension from 2'-3" (.7 m) to 4'-3" (1.3 m). A 3-rail metal railing is attached to a 9 in. (229 mm) high by 24 in. (610 mm) wide brush curb. While this railing configuration does not provide the same level of vehicle safety as the concrete redirectional barrier on the existing structure, it does meet existing AASHTO criteria. It has been used to illustrate a method of widening an existing bridge deck through an increase in the allowable deck overhang.

A cross section showing the proposed deck replacement is shown in Figure 21. Both a precast modular slab and a conventional cast-in-place slab are shown. The latter is shown only for the purpose of comparison to be developed in Chapter V. Detailed plans are shown for the modular slab system in Figure 22.

The prestressed precast deck slab modular element is placed in two sections, each comprising approximately one-half of the total structure width. The first half is placed just short of the centerline of the structure, resting on the flange of the beam. The existing deck is saw-cut and removed so that the edge of the existing deck also rests on the center beam, providing support for the existing structural slab. See Figure 22, Section G-G for a detail of this center connection.

The interior modules are 8 ft. (2.4 m) wide, measured perpendicular to the skew of the bridge. The modules on each end vary in width to make up the overall bridge length. The individual units weigh about 11 tons (10,000 kg).

Mild reinforcement is provided in the slabs for distribution, temperature, and shrinkage. The bottom mild reinforcement consists of a welded wire fabric mat. The top longitudinal reinforcement is #4 bars, epoxy-coated.

Longitudinal post-tensioning ducts are provided at intervals between the two levels of prestressing wire to permit post-tensioning of the entire deck in the longitudinal direction.

Slab Design. Current AASHTO specifications call for a minimum slab thickness based on the following formula:

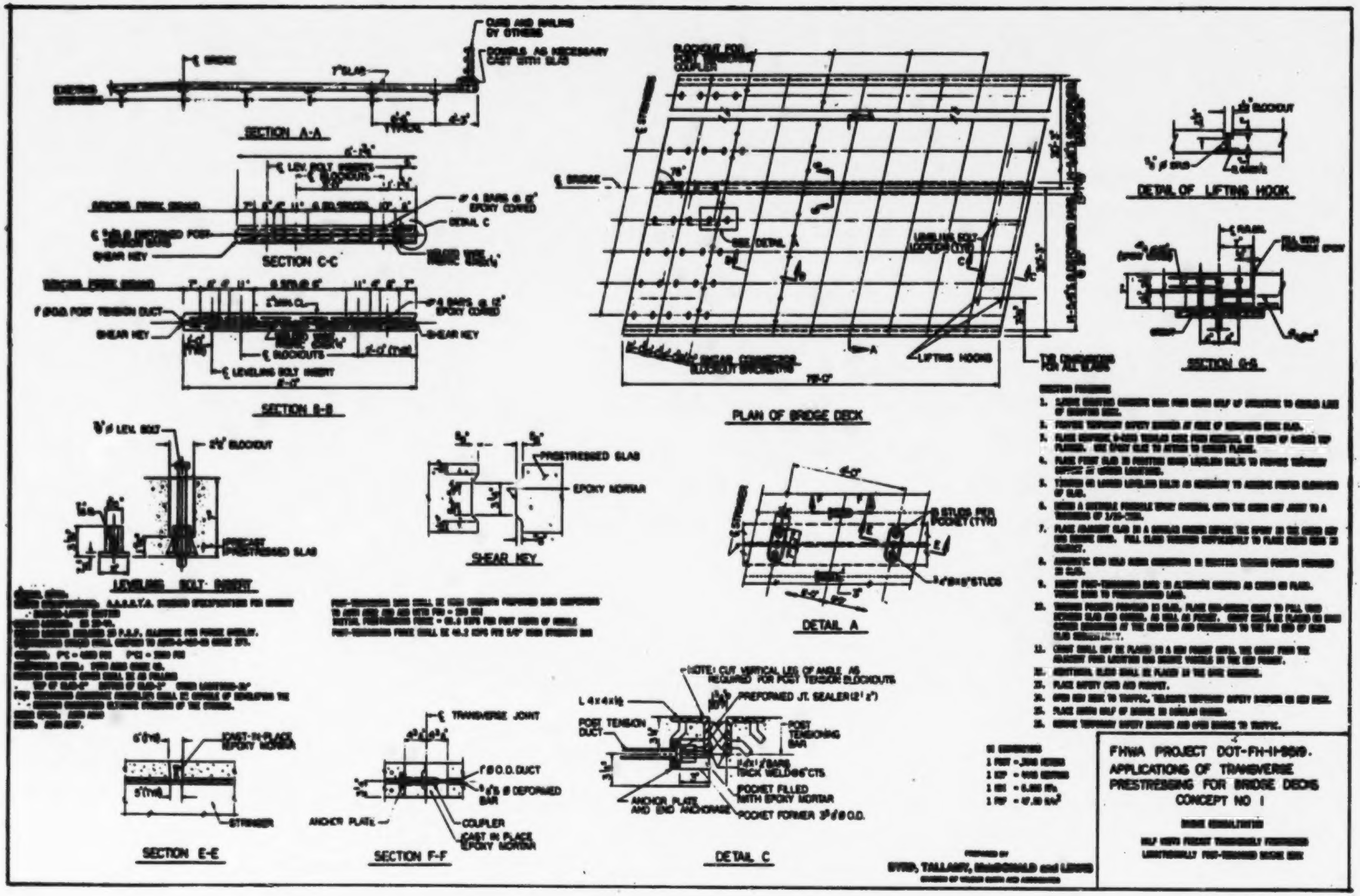
SECTION A-A
SECTION C-C
SECTION B-B
PLAN OF BRIDGE DECK
DETAIL OF LIFTING HOOK
SECTION G-G
LEVELING BOLT INSERT
SHEAR KEY
DETAIL A
SECTION E-E
SECTION F-F
DETAIL C
CURB AND RAILING BY OTHERS
DOWELS AS NECESSARY CAST WITH SLAB
7"SLAB
SHEAR KEY
DETAIL C
℄ BRIDGE
SEE DETAIL A
LEVELING BOLT
LIFTING HOOKS
PRESTRESSED SLAB
EPOXY MORTAR
PRECAST PRESTRESSED SLAB
℄ LEVELING BOLT INSERT
℄ BLOCKOUTS
2½" BLOCKOUT
STUDS PER POCKET (TYP)
NOTE: CUT VERTICAL LEG OF ANGLE AS REQUIRED FOR POST TENSION BLOCKOUTS
PREFORMED JT. SEALER(2'2")
L 4x4x½
POST TENSION DUCT
POST TENSIONING BAR
POCKET FILLED WITH EPOXY MORTAR
ANCHOR PLATE AND END ANCHORAGE
POCKET FORMER
TRANSVERSE JOINT
COUPLER
ANCHOR PLATE
CAST IN PLACE EPOXY MORTAR
CAST-IN-PLACE EPOXY MORTAR
STRINGER
FHWA PROJECT DOT-FH-11-9519.
APPLICATIONS OF TRANSVERSE
PRESTRESSING FOR BRIDGE DECKS
CONCEPT NO 1

Figure 22 Concept No. 1 - Bridge Rehabilitation

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$$D = \frac{S + 10}{30} \geqslant .542 \text{ ft.}$$

$$D = \frac{S + 3.05}{30} \geqslant .165 \text{ m}$$

A thickness of 6-1/2 in. (165 mm) is therefore the minimum allowed. To accommodate a single longitudinal post-tensioning duct in the center of the slab, it is necessary to place the prestressing wire in two rows symmetrically about the centerline of the slab depth. Using the minimum clearance requirements of 2 in. (51 mm) for top steel cover, a depth of 7 in. (178 mm) is required. The prestressing steel is placed concentric with the centerline of the slab depth.

Interior bending moments are determined by using standard formulas for main reinforcement perpendicular to traffic. Because the slab is continuous over more than three supports, a continuity factor of 0.8 is used. For purposes of this concept, an allowance of 30 psf (137 kg/m^2) is provided for application of a future wearing surface. Design calculations are based on standard AASHTO HS 20-44 truck loads.

The proposed overhang dimension of 4'-3" (1.3 m) is greater than normally considered in new design work. It is therefore necessary to compare the cantilever moments with the interior moments. Live load moments are determined by the formula

$$\text{Moment per foot (m) of slab} = \frac{P}{E} \text{ x ft. lbs. (Nm)}$$

$$\text{where } E = 0.8X + 3.75\ (0.8X + 1.143).$$

Dead load moments are determined using standard formulas for uniform loads on cantilever beams. Comparing these moments indicates that the cantilever does control the slab design, resulting in an increase of 25% in the applied moments.

The modular slab is designed to permit no tension under live load. This eliminates the need for bonded mild reinforcement in the precompressed tensile area which would require the placement of transverse steel in both the top and bottom of the slab over its entire length.

The determination of the required prestressing force is obtained by equating the precompression stress level to the maximum fiber stress produced by application of all design loads.

Prestress losses are estimated to be 20% in accordance with AASHTO specifications. The design is not considered to be unusual, nor are the prestress levels outside the range of normal design parameters. The value of 20% for losses is, therefore, a realistic estimate.

Longitudinal post-tensioning of skewed bridge deck modules, as in this concept, creates force components normal and parallel to the individual units. This force is used to decrease the need for transverse prestressing steel in the slab. The longitudinal post-tensioning force is set at a value of 200 psi (.38 MPa). The decrease in required transverse prestressing force is calculated using the formula

$$P_i \times A \text{ Sin } (a)$$

where P_i = Longitudinal post-tensioning force in lb/sq. in. (MPa)
A = Cross-section area per ft. (m) width of slab
(a) = Skew angle of deck slab.

This force is subtracted from the initial prestress force. The initial prestress force required is calculated using the formula

$$P_i = \frac{P}{1 - \%\text{loss}}$$

This is the initial prestress force required to offset the final moments applied to the slab. The number of prestressing strands required is calculated by dividing the prestressing load (P) by the allowable initial prestressing force per strand.

In this example a total of 26 7/16"Ø (11 mm) strands of 270 KSI (1861.58 MPa) ultimate strength are required. These strands are placed in two rows of 13 each, concentric with the slab depth.

Based upon studies by Martin and Scott(7) the average development length for a 7/16 in. (11 mm) prestressing strand is 33 in. (838 mm). This is less than the distance to the point of maximum moment, which is 47" (1194 mm). The development length is therefore adequate to transfer the full prestressing force into the deck module before the point of maximum moment is reached.

The longitudinal post-tensioning is applied symmetrically across the deck cross-section. A total of 28 5/8"Ø (16 mm) high-strength deformed bars are required for the total bridge cross-section.

Compressive strength of concrete at 28 days is determined assuming that the openings provided for shear connectors and other inserts are subtracted from the area available to resist the initial prestressing force. Because these openings are filled only after all prestressing forces have been applied, the initial precompression of the remaining concrete cross-section is increased in proportion to the percentage of the total section loss. This increases the final concrete stress from 3500 to 4000 psi (24.13 to 27.58 MPa). Calculations are provided in Appendix A.

Deck Details. The transverse joints between slabs are male and female configuration. This joint configuration was selected to provide a simple yet tight joint capable of transferring vertical shear between units. A coating of pourable epoxy is placed on the leading edge of the slab already in position, and while the epoxy is in a plastic state, the adjacent module is placed so that the joints come in contact as evidenced by the compression and squeezing out of the epoxy.

The longitudinal joint consists of an overlapping configuration occurring directly above the center beam. A pourable epoxy is placed on the joint to provide bond between the slabs (see Figure 22, Section G-G).

The cross-slope of the deck slab is provided through placement of the individual modules on each half of the bridge deck to pre-determined elevations at the median and fascia beams. Due to the lap joint at the center of the bridge, the break in the crown will occur at the edge of the lap joint. The individual joint faces are fabricated normal and parallel to the deck slab module. A 1/2 in. (13 mm) clearance is provided between the individual slabs at the center joint to accommodate variations resulting from the cross-slope.

Four leveling bolts are provided for vertical alignment adjustments and temporary support for each slab unit. Once the first slab is placed and aligned vertically by adjusting the projection of the bolt end below the bottom of the slab, additional slabs are placed and aligned in the same manner. At beam locations where leveling bolts are not used, steel or neoprene bearing pads are wedged between the slab and support beam to provide temporary bearing capacity.

Bearing Requirements. The area between the top of the support girder and the bottom of the precast deck modules must be filled with a non-compressible material capable of transferring the vertical loads from the weight of the deck and the applied live loads into the support girder. This material must be placed in a liquid state since it will not be placed until the deck modules have been positioned on the supports. Because means have been used to temporarily carry the dead load reactions into the girder through shim plates or leveling bolts, the side forms for this application can be flexible. Tubular neoprene which can conform to irregularities in the closure dimension is provided.

The modular deck is attached to the wide flange beams by shear studs welded to the top flange through 3 in. by 14 in. (76 mm x 356 mm) cutouts preformed in the deck module during fabrication. Detail A on Figure 22 shows the configurations of these cutouts and the shear studs.

Construction Sequence. The construction sequence for placement of the modular deck system is as follows:

1. Provide barricades, warning signs and other necessary traffic safety devices necessary to relocate all traffic onto portion of deck not being rehabilitated in Phase 1 of the project.
2. Saw cut existing bridge deck at centerline of bridge cross-section and, using suitable equipment, begin removal of one-half of width of existing deck. Removal will commence from one end to the other, using the existing deck as a staging area for equipment.
3. Placement of the modular deck units may proceed as the original deck is being removed, or may be accomplished as an independent operation after removal of the deteriorated deck.
4. The top flanges of the existing beams are thoroughly cleaned by sandblasting and the neoprene O-ring side form strips are attached to the edges of the flanges using epoxy glue.
5. Place first deck module in position at end of bridge, using the leveling bolts to raise or lower the slab to its proper relative elevation. These leveling bolts can be turned down before placement so that the initial elevation of the module is above its

final anticipated elevation. Shim pads can then be placed beneath the edge of the module at intermediate beam locations, and the leveling bolts used to drop the module into its correct elevation while at the same time providing proper temporary bearing under each deck module.

6. Brush a suitable pourable epoxy mortar on the leading edge of the module just positioned.
7. While the epoxy mortar is still in a plastic state, place the second deck module as provided in step 5 above. The new slab must be aligned both vertically and horizontally. The post-tensioning ducts must be properly aligned and the shear joint must be tight to within a tolerance of 1/16 in.(1.6 mm) to guarantee bonding between the concrete and the epoxy mortar.
8. Weld shear studs to the steel beams through the preformed pockets provided in the slabs.
9. Place post-tensioning bars through alternate post-tensioning ducts provided. Using hydraulic jacks, post-tension each bar to initial prestress force of 23.1 tons (21 t). The bars should be stressed in a symmetrical manner about the centerline of the deck module.
10. Using the preformed pockets in the deck module, place non-shrink grout in the space between the deck module and the support girder. Care must be exercised to assure that this cavity is completely filled with grout. The placement of grout should commence from one end of the slab, forward. Grout should not be placed in an adjacent preformed pocket until grout from the adjacent pour is visible in the new pocket.
11. Using this same sequence, complete the entire bridge deck width.
12. Because the post-tensioning bars have been stressed in alternating patterns throughout, after placing the final deck module, all the post-tensioning bars must be stressed to the initial force.
13. The post-tensioning ducts will be filled with suitable grout, pressure injected through grout ports provided in the slab as needed.

14. Place curbs and rail units.

15. Divert traffic to the new deck slab and follow a similar procedure for the second side.

16. While placing the slabs on the second half of the bridge, the epoxy must be placed on the longitudinal joint at the same time it is being placed in the transverse joint.

DESIGN CONCEPT NUMBER 2 - BRIDGE REHABILITATION

Design Concept Number 2 illustrates the use of longitudinally prestressed precast structural bridge deck elements on an existing truss bridge, using only the floor beams as deck supports. The concept is also applicable for bridges built with a girder-floor beam framing system.

Critical areas of concern are:

1. Speed of Erection. For purposes of this concept, it is assumed that the bridge is serving a large population area and closing the bridge would create a major inconvenience to the traveling public. The structure crosses a major stream. The lack of adequate roadway width on the bridge requires the bridge to be closed to traffic during the reconstruction. All traffic presently using the facility must be detoured to one of two alternate bridges. This will create traffic delays on the alternate routes during the entire construction period.

2. Dead Load Weight. Although there is some capacity for live load in the existing floor beams and truss members, excessive dead load increases will place the structure in danger of failure under standard HS20-44 highway loadings.

Existing Structure

The existing structure is a single span through truss bridge. The bridge deck is supported on floor beams and stringers. The length of the bridge is 95'-6" (29 m) between bearings. The structure is designed for noncomposite action. The original structural concrete deck slab varies in thickness from 6 in. (152 mm) at the curb line to 9 in. (229 mm) at the centerline of the roadway.

The existing roadway is 24'-0" (7.3 m) wide providing two 12-ft. (4 m) traffic lanes with no shoulders. Concrete brush curb and safety parapets 1'-9" (.53 m) wide complete the deck width for a total of 27'-6" (8.4 m) from out to out of parapet. The seven existing steel support beams are spaced at 4 ft. (1.2 m) on center, with an overhang of 1'-9" (.5 m) from the centerline of the fascia beam to the face of the concrete parapet. The bridge deck surface incorporates a 1/4 in./ft. (21 mm/m) cross-slope downward each side of the centerline. The stringers are positioned at a constant elevation across the deck width, necessitating the variation in deck slab thickness from 6 in. (152 mm) to 9 in. (229 mm) to accommodate the cross-slope.

Deterioration over the life of the structure has decreased the capacity of the existing stringers below that required to carry present design loads. Therefore, the stringers must be strengthened,or replaced, or a slab capable of spanning between floor beams must be employed.

Proposed Deck Replacement

The proposed bridge deck spans 15'-11" (4.9 m) between floor beams. The concrete brush curb and parapet are replaced by a more open 3-rail metal railing attached to a 9 in. (229 mm) high brush curb.

A cross-section showing the proposed deck replacement is shown in Figure 23. Both a precast modular slab and a conventional cast-in-place slab are shown. The latter is shown only for purposes of comparison to be developed in Chapter V. Detailed plans are shown for the modular system in Figure 24.

The prestressed precast deck slab modular elements are placed in two sections, each comprising approximately one-half of the total structure length. The individual modules are 6'-10 1/2" (2.1 m) wide. The individual units weigh 20 tons (18,180 kg).

Mild reinforcement is provided in the slabs for distribution and for temperature and shrinkage. The bottom mild reinforcement consists of a

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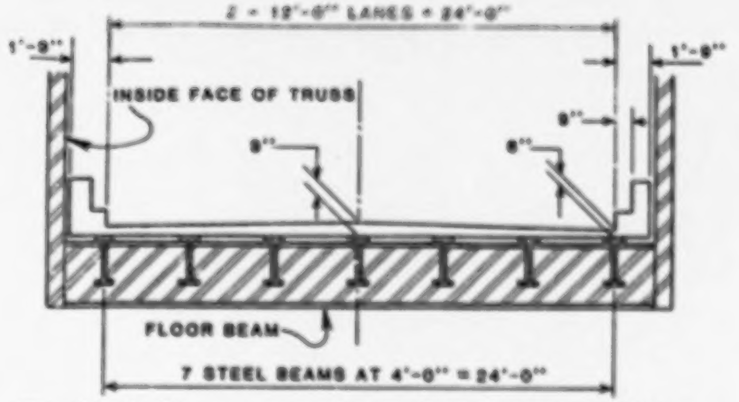
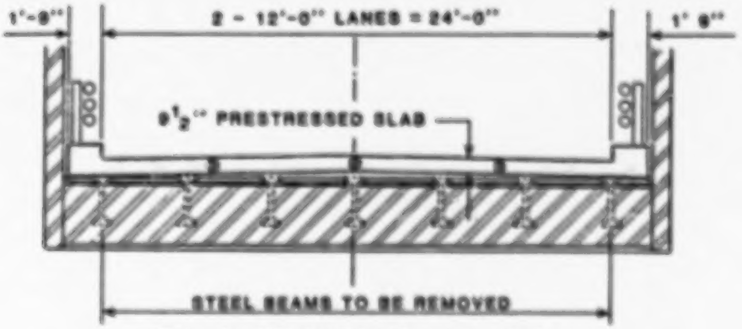
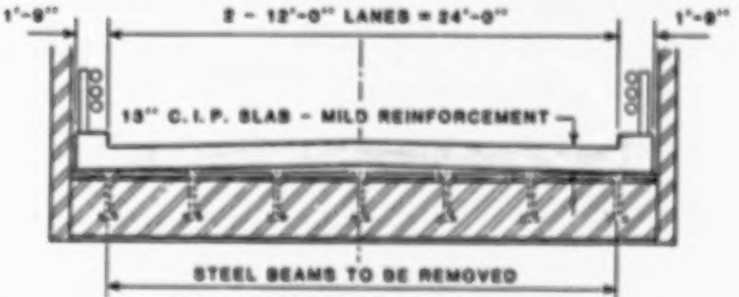

2 - 12'-0" LANES = 24'-0"
1'-9"
1'-9"
INSIDE FACE OF TRUSS
9"
9"
9"
FLOOR BEAM
7 STEEL BEAMS AT 4'-0" = 24'-0"

Typical Cross-section of Existing Bridge


1'-9"
2 - 12'-0" LANES = 24'-0"
1'-9"
9½" PRESTRESSED SLAB
STEEL BEAMS TO BE REMOVED

Deck Slab Replacement

Transversely Prestressed Precast Deck


1'-9"
2 - 12'-0" LANES = 24'-0"
1'-9"
13" C.I.P. SLAB - MILD REINFORCEMENT
STEEL BEAMS TO BE REMOVED

Deck Slab Replacement

C.I.P. SLAB - Mild Reinforcement

Figure 23. Typical Cross-Section
Design Concept Number 2

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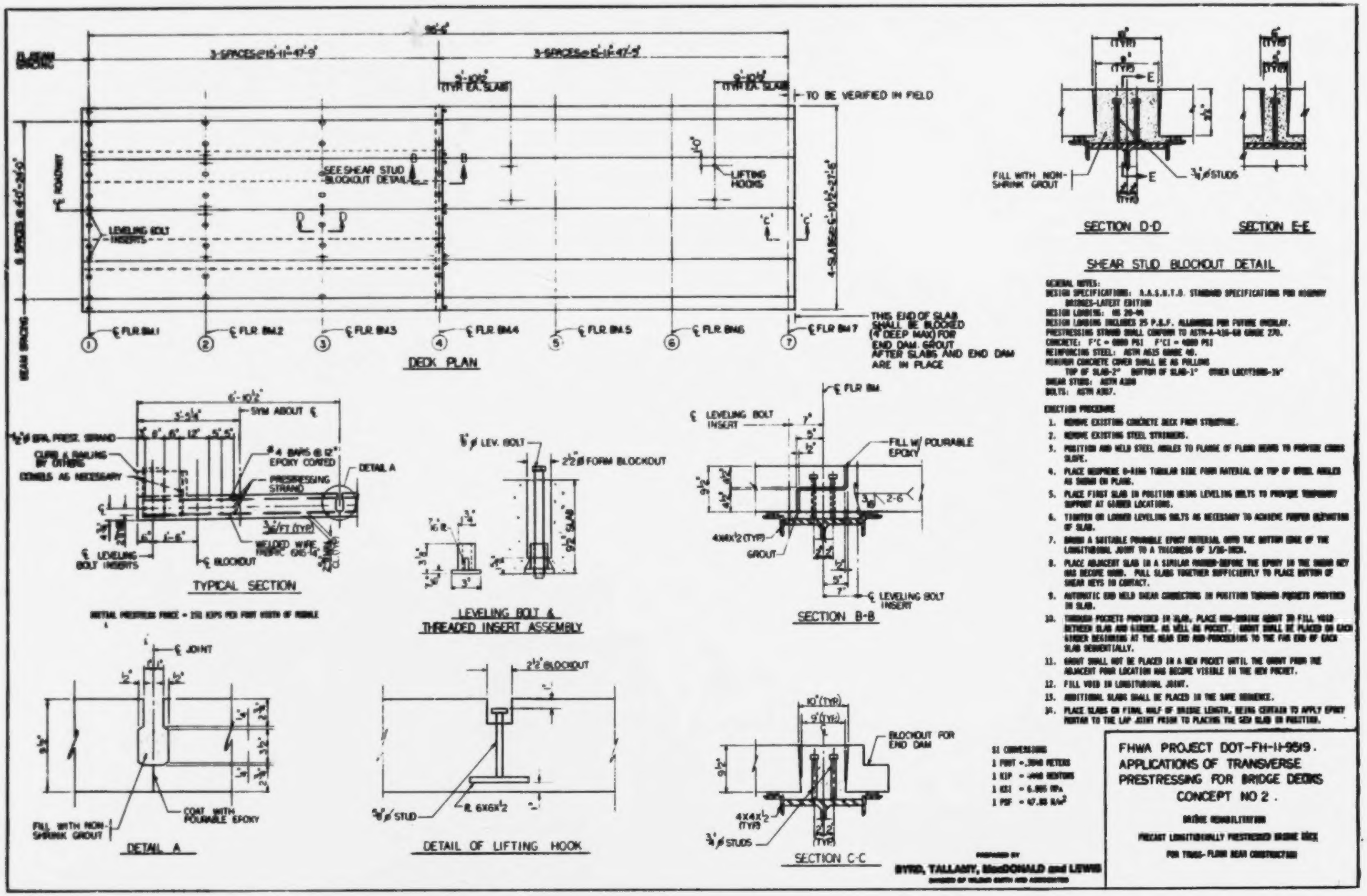
DECK PLAN
3-SPACES@15'-11"=47'-9"
TO BE VERIFIED IN FIELD
SEE SHEAR STUD BLOCKOUT DETAIL
LIFTING HOOKS
LEVELING BOLT INSERTS
℄ FLR BM 1
℄ FLR BM 2
℄ FLR BM 3
℄ FLR BM 4
℄ FLR BM 5
℄ FLR BM 6
℄ FLR BM 7
THIS END OF SLAB SHALL BE BLOCKED (4" DEEP MAX) FOR END DAM GROUT AFTER SLABS AND END DAM ARE IN PLACE
SECTION D-D
SECTION E-E
FILL WITH NON-SHRINK GROUT
3/4"ø STUDS
SHEAR STUD BLOCKOUT DETAIL
TYPICAL SECTION
SYM ABOUT ℄
PRESTRESSING STRAND
EPOXY COATED
CURB & RAILING BY OTHERS
DOWELS AS NECESSARY
DETAIL A
WELDED WIRE FABRIC
℄ LEVELING BOLT INSERTS
℄ BLOCKOUT
LEVELING BOLT & THREADED INSERT ASSEMBLY
LEV. BOLT
FORM BLOCKOUT
SLAB
SECTION B-B
℄ FLR BM
℄ LEVELING BOLT INSERT
FILL W/ POURABLE EPOXY
GROUT
DETAIL A
℄ JOINT
COAT WITH POURABLE EPOXY
FILL WITH NON-SHRINK GROUT
DETAIL OF LIFTING HOOK
2½" BLOCKOUT
5/8"ø STUD
SECTION C-C
BLOCKOUT FOR END DAM
3/4"ø STUDS
FHWA PROJECT DOT-FH-11-9519.
APPLICATIONS OF TRANSVERSE PRESTRESSING FOR BRIDGE DECKS
CONCEPT NO 2.
BYRD, TALLAMY, MacDONALD and LEWIS

Figure 24. Concept No.2 - Bridge Rehabilitation

welded wire fabric mat. The top transverse reinforcement is epoxy-coated.

Slab Design - This design concept assumes that the existing stringers are to be removed and a longitudinally reinforced deck slab is designed as a replacement. Each slab module is a 3-span continuous unit with a width equal to one-fourth of the total deck width. Figure 23 shows a typical cross-section of the proposed deck configuration.

AASHTO specifications provide for minimum slab thickness recommendations based upon the same formula given in design concept number 1 which yields a depth of .86 ft. (261 mm). The depth to span ratio recommended for prestressed concrete longitudinal bridge members is .04 which yields a depth of .64 ft. (190 mm). Providing minimum clearance of 2 in. (51 mm) for top steel cover and using three rows of prestressing with the strands placed at 2 in. (51 mm) center to center, a 9-1/2 in.(241 mm) is required.

Bending moments in continuous spans are determined by analysis using existing tables for multi-span continuous beams. The distribution of wheel loads, in accordance with AASHTO, is given by the formula

$$E = 4 + 0.06S \leq 7 \text{ ft.}$$

$$(E = 1.219 + 0.06S) \leq 2.134 \text{ m}$$

This formula yields a distribution of 4.91 ft. (1.5 m). Bending moments are determined using co-efficients found in the publication "Moments Shears and Reactions for Continuous Highway Bridges" produced by the American Institute of Steel Construction. The computations for dead load, live load and impact moments are given in Appendix A. An allowance of 25 psf (122 kg/m^2) is provided for future wearing surface.

The modular slab is designed to permit no tension under live load. This eliminates the need for mild reinforcement in the precompressed tensile area.

Prestress losses as in concept number 1 are estimated to be 20% in accordance with AASHTO specifications. The initial prestress force required is calculated using the formula given in Design Concept Number 1. In this example, a total of 36 1/2 in. (12.7 mm) strands of 270 KSI (186.58 MPa) ultimate strength are required. These strands are placed in three rows of 12 each, concentric with the slab depth.

Compressive strength of concrete at 28 days is determined assuming that the openings provided for shear connectors and other inserts will be subtracted from the area available to resist the initial prestressing force. Because these openings are filled only after all prestressing forces have been applied, the initial precompression of the remaining concrete cross-section is increased in proportion to the percentage of the total section loss. This increases the final concrete stress from 5200 to 6000 psi (35.85 to 41.37 MPa). Calculations are provided in Appendix A.

Deck Details - The longitudinal joints between slabs are female-female configuration. This joint configuration was selected to provide a simple joint capable of transferring vertical shear between units. A coating of pourable epoxy is placed on the bottom leading edge of the slab already in position, and while the epoxy is in a plastic state, the adjacent module is placed so that the bottom of the joint comes in contact as evidenced by the compression and squeezing out of the epoxy. A high strength, non-shrink grout is later poured into the joint from above, producing a double shear key.

The cross-slope of the deck slab is provided during placement of the individual modules on each half of the bridge deck to predetermined elevations. The break in the crown will occur at the centerline of the bridge. The bottom portion of the shear key at the center joint will touch only at the bottom edge. An extra heavy application of epoxy to this part of the joint will fill the small gap at the bottom. The opening at the top of the slab will be increased by approximately 1/4 in. (6.4 mm).

Using this joint permits easy vertical alignment of the surface of the slab at the joint locations. Minor inconsistencies in slab thickness and joint dimensions can be accommodated.

The transverse joint consists of an overlapping type occuring directly above the center floor beam. The joint is centered on the floor beam to provide adequate support for the deck module. A pourable epoxy is placed on the joint to provide bond between the slabs (See Section B-B on Figure 24).

Leveling bolts are provided at each floor beam location for vertical alignment and temporary support for the slab units. Once the first slab has been placed and aligned vertically by adjusting the projection of the bolt end below the bottom of the slab, additional slabs are aligned in the same manner. Because the floor beam top flange is level across the width of the bridge, each succeeding leveling bolt from the outside of the bridge deck toward the center at the crown of the roadway will require a greater bolt projection from the bottom of the slab.

Bearing Requirements - The area between the top of the floor beam and the bottom of the precast deck modules is filled with a non-compressible material capable of transferring the vertical loads from the weight of the deck and the applied live loads to the floor beam.

The leveling bolts are used to temporarily support the dead load of the slab. The floor beams are level across the deck width. In order to decrease the large variation in the closure dimension of the space between the deck module and floor beam top flange, steel angles are welded to the edges of the top flange (See Figure 24, Section B-B).

A tubular neoprene is used in a manner similar to its use on Concept number 1. The tubular neoprene material will conform to any irregularities between the deck module surface and the steel angle.

The modular deck is attached to the floor beams by use of shear studs welded to the top flange through 6 in. x 10 in.(152x254 mm) cutouts preformed in the deck module during fabrication. Section D-D and E-E on Figure 24 show the configuration of the cutouts and the shear studs.

Construction Sequence - The construction sequence for placement of the modular deck system is as follows:

1. Provide barricades, warning signs and other necessary traffic safety devices necessary to detour all traffic onto alternate routes during rehabilitation.
2. Saw cut existing bridge deck and, using suitable equipment, begin removal of existing deck. Removal will commence from one end to the other, using the existing deck as a staging area for equipment.
3. Remove existing stringers from floor beams by flame cutting stringer webs or by chipping and removal of existing web connectors.

4. Placement of the modular deck units may proceed after the original deck and stringers have been removed from the first half of the bridge, or may be accomplished as an independent operation after removal of the deteriorated deck.

5. Thoroughly clean the top flange of the existing floor beams by sandblasting and weld the steel angle side forms to the edges of the floor beams.

6. Glue the neoprene O-ring to the top of the horizontal leg of the side form angles using epoxy glue.

7. Place first deck module in position using the leveling bolts to raise or lower the slab to its proper elevation. These leveling bolts can be turned down before placement so that the initial elevation of the module is above its final anticipated elevation.

8. Brush a pourable epoxy mortar on the bottom longitudinal edge of the module just positioned.

9. While the epoxy mortar is still in a plastic state, place the second deck module as provided in step 5 above. The new slab must be aligned both vertically and horizontally. The shear joint must be tight to within a tolerance of 1/16 in.(1.6 mm) to guarantee bonding between the concrete and the epoxy mortar at the base of the joint.

10. Weld shear studs to the steel beams through the preformed pockets provided in the slabs.

11. Using the preformed pockets in the deck module, place non-shrink grout in the space between the deck module and the support girder. Care must be exercised to assure that this cavity is completely filled with grout. The placement of grout should commence from one side of the slab to the other. Grout should not be placed in a new preformed pocket until grout from the adjacent pour is visible in the new pocket.

12. Fill void in longitudinal joint using non-shrink grout to establish the shear key. Strike all mortar off flush with the deck surface.

13. Using the same sequence, complete the entire bridge deck width.

14. While placing the slabs on the second half of the bridge, the epoxy must be placed on the transverse lap joint at the same time it is placed on the longitudinal joint.

DESIGN CONCEPT NUMBER 3 - NEW BRIDGE CONSTRUCTION

Design Concept Number 3 illustrates the applicability of a transversely prestressed precast structural bridge deck system for new bridges as an alternative to conventional construction. The concept also presents a method for providing composite design.

Critical areas of concern are reduced time needed for erection and a reduction in dead load of the slab and superstructure.

Proposed Structure

The proposed structure is a single span precast, prestressed concrete I-beam, multi-girder bridge with an overall length of 65 ft. (19.8 m). The structure is designed to act compositely under superimposed dead loads and live loads. The substructure units are skewed at an angle of 20 degrees.

The roadway is 34 ft. (10.4 m) wide including two 12-ft. (3.7 m) traffic lanes and two 5-ft. (1.5 m) shoulders. The 9 in. (229 mm) safety curbs are 16 in.(406 mm) wide. A 3-rail metal railing is mounted on the safety curb. The overall width of the structure is 36'-8" (11.2 m). There are four precast, prestressed concrete I-beam support beams spaced at 9'-6" (2.9 m) center to center. The slab overhang at the fascia is 4'-1" (1.2 m) The structural bridge deck is level. The roadway crown is incorporated in the placement of the waterproof overlay.

A typical cross-section of the proposed bridge is shown in Figure 25. Both a conventional cast-in-place concrete deck and a modular precast deck are shown. The former is included only for purposes of comparison developed in Chapter V. Detailed plans are shown in Figure 26 for the modular system.

Modular Deck

A simple prestressed precast deck slab module covers the full width of the bridge. Seven of the eight modules are 8 ft. (2.4 m) wide, measured

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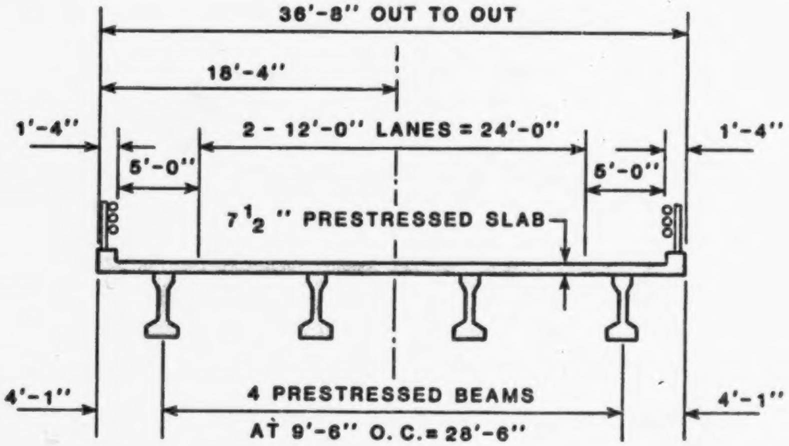
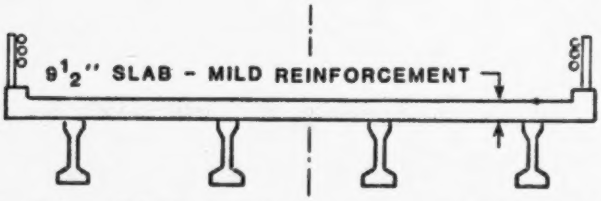
perpendicular to the skew of the bridge. The final module is of sufficient width to make up the overall bridge length. The individual units weigh about 15 tons (13,300 kg).

Mild reinforcement is provided in the slabs for distribution and for temperature and shrinkage. The bottom mild reinforcement consists of a welded wire fabric net. The top longitudinal reinforcement is #4 bars epoxy-coated.


36'-8" OUT TO OUT
18'-4"
1'-4"
2 - 12'-0" LANES = 24'-0"
1'-4"
5'-0"
5'-0"
7½ " PRESTRESSED SLAB
4'-1"
4 PRESTRESSED BEAMS
AT 9'-6" O.C. = 28'-6"
4'-1"

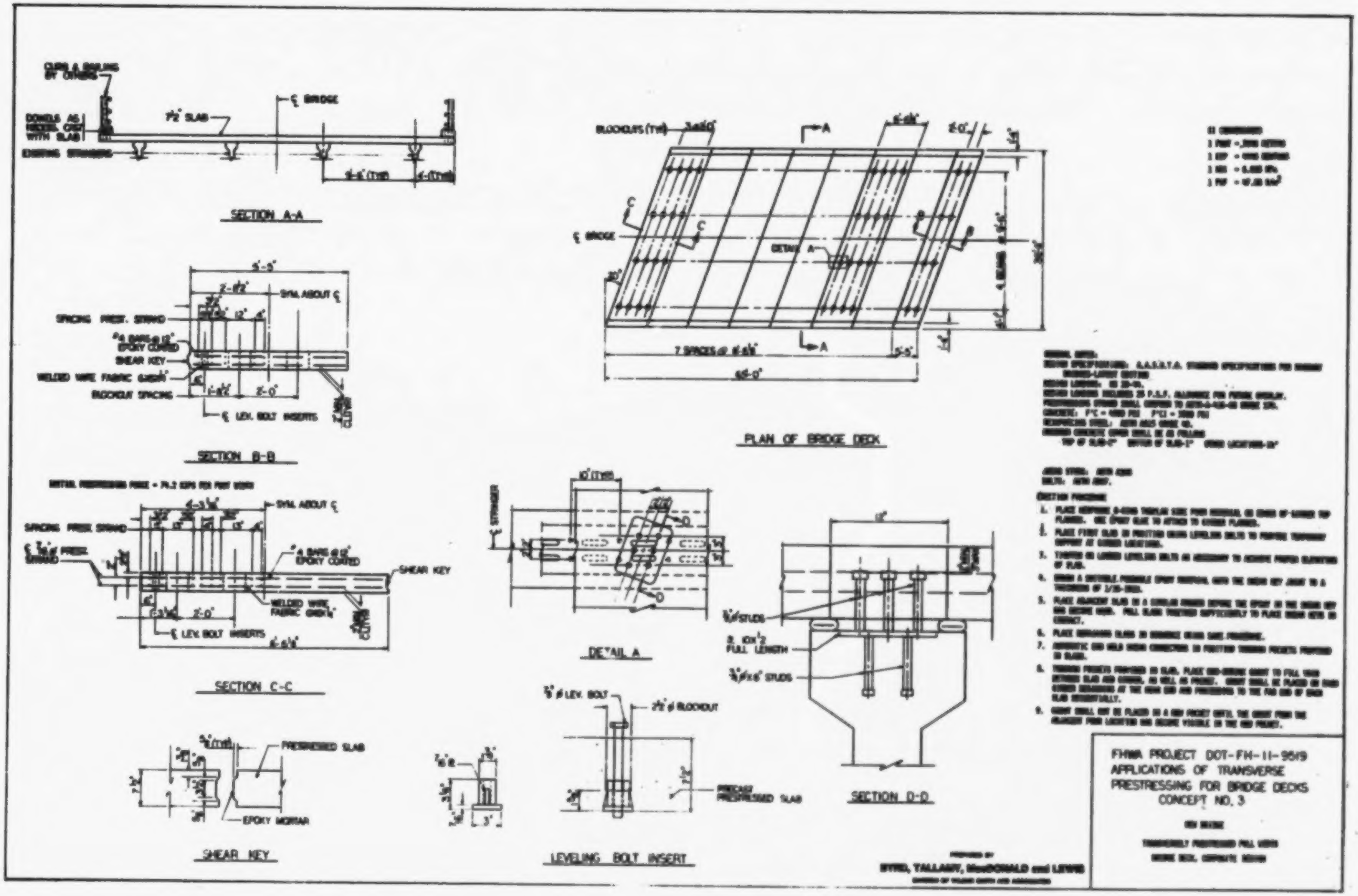
Transversely Prestressed Precast Slab


9½" SLAB - MILD REINFORCEMENT

C.I.P. Slab - Mild Reinforcement

Figure 25. Typical Cross-Section Design Concept Number 3

Slab Design - In the preliminary design stage for this structure, the overhang dimension and subsequent girder spacing were varied to maximize the efficiency of the prestressing, equating the maximum cantilever moment to the corresponding interior moment as determined in accordance with AASHTO specifications. This preliminary design resulted in a 9'-6" (2.9 m) girder spacing with a 4'-1" (1.2 m) deck cantilever.


CURB & RAILING BY OTHERS
DOWELS AS REQ'D. CAST WITH SLAB
EXISTING STRINGERS
7½" SLAB
℄ BRIDGE
SECTION A-A
5'-5"
SYM. ABOUT ℄
SPACING PREST. STRAND
SHEAR KEY
WELDED WIRE FABRIC
BLOCKOUT SPACING
℄ LEV. BOLT INSERTS
SECTION B-B
SPACING PREST. STRAND
SYM. ABOUT ℄
EPOXY COATED
SHEAR KEY
WELDED WIRE FABRIC
℄ LEV. BOLT INSERTS
SECTION C-C
BLOCKOUTS (TYP.)
℄ BRIDGE
DETAIL A
7 SPACES @
PLAN OF BRIDGE DECK
℄ STRINGER
10" (TYP.)
DETAIL A
12"
FULL LENGTH
STUDS
SECTION D-D
PRESTRESSED SLAB
EPOXY MORTAR
SHEAR KEY
LEV. BOLT
BLOCKOUT
PRECAST PRESTRESSED SLAB
LEVELING BOLT INSERT

FHWA PROJECT DOT-FH-11-9519
APPLICATIONS OF TRANSVERSE
PRESTRESSING FOR BRIDGE DECKS
CONCEPT NO. 3

PREPARED BY
BYRD, TALLAMY, MacDONALD and LEWIS

The minimum slab thickness based upon the AASHTO formula is .61 ft.(.19 m). The depth used is 7-1/2 in.(191 mm). The prestressing strands are placed in two rows 2 in.(51 mm) apart vertically, concentric with the centerline of the slab depth. This provides a minimum clear cover over the prestressing strands of 2-1/2 in. (64 mm).

Interior bending moments are determined by using formulas developed for main reinforcement perpendicular to traffic. Because the slab is continuous over more than 3 supports, a continuity factor of 0.8 is used. For purposes of this concept an allowance of 25 psf (122 Kg/m^2) is provided for application of a future wearing surface. Design calculations are based on standard AASHTO HS 20-44 truck loads. A check of the cantilever moment indicates that the cantilever does not control the slab design.

The slab is designed for no tension under live load, eliminating the need for mild reinforcement in the precompressed tensile area.

The determination of the prestressing force required is obtained by equating the precompression stress level to the maximum fiber stress produced by application of all design loads. Prestress losses are estimated at 20% in concurrence with AASHTO specifications as used in concepts 1 and 2.

The initial prestress force required is calculated using the same formula given in Design Concept Number 1. The number of prestressing strands required is 32 7/16"Ø (11 mm) strands of 270 KSI (1861.58 MPa) ultimate strength. These are placed in two rows of 16 each.

The average development length for a 7/16 in. (11 mm) prestressing strand is 33 in. (838 mm). This is less than the distance to the point of maximum cantilever moment, which is 39" (991 mm). The development length is therefore adequate to transfer the full prestressing force into the deck module before the point of maximum moment is reached.

Compressive strength of concrete at 28 days is determined assuming that the openings provided for shear connectors and other inserts will be subtracted from the area available to resist the initial prestressing force. Because these openings are filled only after all prestressing forces have been applied, the initial precompression of the remaining concrete cross-section is increased in proportion to the percentage of the total section loss. This increases the final concrete stress from 3000 to 4500 psi (20.68 to 31.02 MPa).

Composite action is developed through shear connectors field welded to a steel plate embedded in the top flange of the precast girder. It is assumed that no bond is developed between the beam and the slab.

Design computations for the concept are given in Appendix A.

Deck Details - The transverse joints between slabs are male-female type (See the shear key detail in Figure 26). This was selected to provide a simple yet tight joint capable of transferring vertical shear between units. A coating of pourable epoxy is placed on the leading edge of the slab already in position, and while the epoxy is in a plastic state, the adjacent module is placed so that the joints come in contact as evidenced by the compression and squeezing out of the epoxy.

Leveling bolts are provided for vertical alignment and temporary support for the slab units. Two such bolts at each girder location are sufficient to support each slab. The alignment and adjustment of the slabs is completed in a manner similar to that in Design Concept Number 1.

Bearing Requirements - The area between the top of the support girder and the bottom of the precast deck modules must be filled with a non-compressible material capable of transferring the vertical loads from the weight of the deck and the applied live loads into the support girder. This material must be placed in a liquid state since it will be placed after the deck modules are positioned on the supports. Leveling bolts are provided for temporary support, and therefore, the side forms need not be rigid. Tubular neoprene is provided which can conform easily to irregularities without requiring the form to be prefabricated to the exact dimension required.

The shear studs are welded to the anchor plate provided in the top flange of the girder through 7 in. by 12 in. (178 x 305 mm) preformed cutouts in the deck module. Slotted holes are cut into the anchor plate to provide additional horizontal shear resistance. Details of this connection are shown in Figure 26.

Construction Sequence - The construction sequence for placement of the modular deck system is as follows:

1. Attach neoprene O-ring side form strips to the edges of the top flanges using epoxy glue.
2. Place first deck module in position at end of bridge, using leveling bolts to raise or lower slab to its proper relative elevation.
3. Brush a suitable pourable epoxy mortar on the leading edge of the module just positioned.
4. While the epoxy mortar is still in a plastic state, place the second deck module as provide in step 3 above. The new slab must be aligned both vertically and horizontally. The shear joint must be tight to within a tolerance of 1/16 in. (1.6 m) to assure bonding between the concrete and the epoxy mortar.
5. Using this same sequence, complete the entire bridge deck width.
6. Weld shear studs to the steel anchor plate through the preformed pockets provided in the slabs.
7. Using the preformed pockets in the deck module, place non-shrink grout in the space between the deck module and the support girder. Care must be exercised to assure that this cavity is completely filled with grout. The placement of grout should commence from one end of the slab, forward. Grout should not be placed in an adjacent preformed pocket until grout from the adjacent pour is visible in the new pocket.

DESIGN CONCEPT NUMBER 4 - NEW BRIDGE CONSTRUCTION

Design Concept Number 4 demonstrates the applicability of using transversely precast structural bridge deck elements to decrease the total number of support elements required by increasing the spacing between girders. This concept also shows a means of developing full continuity between multi-width precast deck modules.

Critical areas of concern are a reduced time needed for erection and a reduction in dead load of the slab and superstructure.

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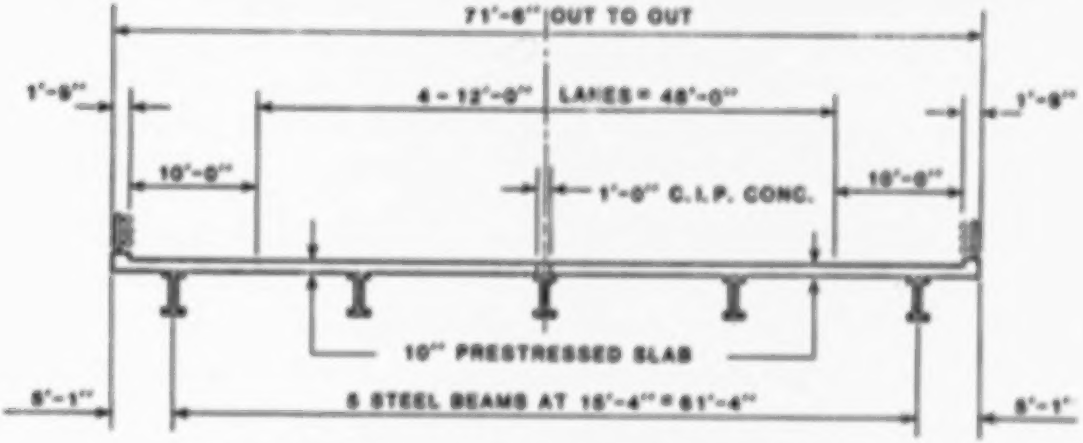
Proposed Structure

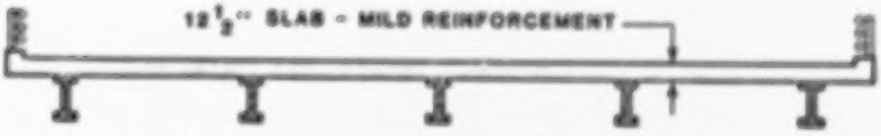
The proposed structure is a single span, multiple steel beam bridge with an overall length of 112 ft. (34.2 m). Composite action to carry live loads and superimposed dead loads is not required.

The roadway is 68 ft. (20.7 m), wide including four 12-ft. (3.7 m) traffic lanes and two 10-ft. (3 m) shoulders. The 9 in. (229 mm) safety curbs are 21 in. (533 mm) wide. A 3-rail metal railing is mounted on the safety curb. The overall width of the structure is 71'-6" (21.8 m). The deck is supported by four steel I-beams spaced at 15'-4" (4.7 m) center to center. The bridge deck surface has a cross-slope of 3/16 in./ft. (16 mm/m) toward the curb on each side of the centerline.

A typical cross-section of the proposed bridge is shown in Figure 27. Both a conventional cast-in-place concrete deck and a modular precast deck are shown. The former is included only for purposes of comparison developed in Chapter V. Detailed plans for the modular system are shown in Figure 28.


71'-6" OUT TO OUT
1'-9"
4 - 12'-0" LANES = 48'-0"
1'-9"
10'-0"
1'-0" C.I.P. CONC.
10'-0"
10" PRESTRESSED SLAB
5'-1"
5 STEEL BEAMS AT 15'-4" = 61'-4"
5'-1"

Transversely Prestressed Precast Slab


12½" SLAB - MILD REINFORCEMENT

C.I.P. Slab - Mild Reinforcement

Figure 27. Typical Cross-Section Design Concept Number 4

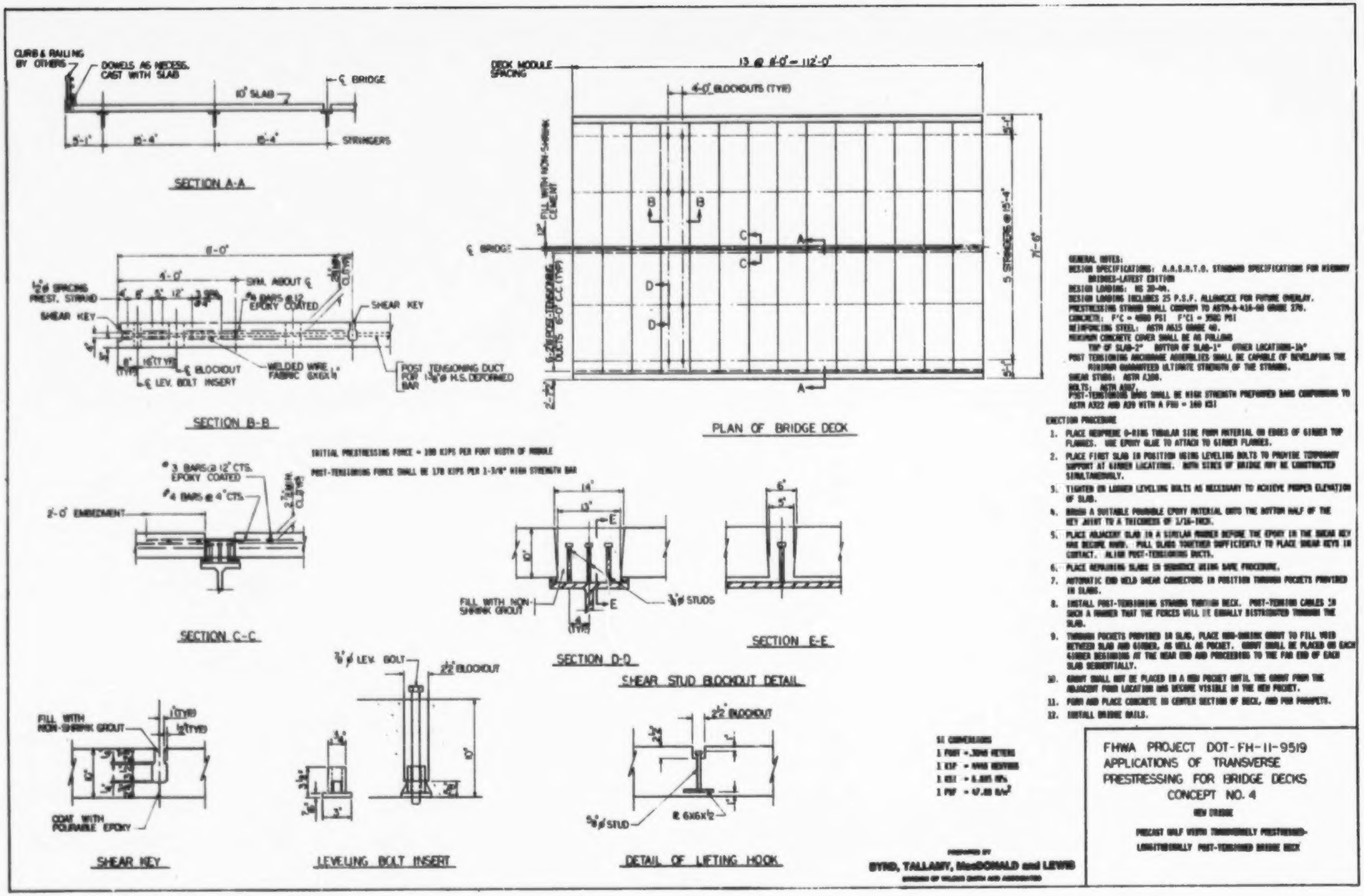

CURB & RAILING BY OTHERS
DOWELS AS NECESS. CAST WITH SLAB
10" SLAB
℄ BRIDGE
5'-1"
15'-4"
15'-4"
STRINGERS
SECTION A-A
11'-0"
4'-0"
SYM. ABOUT ℄
1/2"⌀ SPACING PREST. STRAND
#4 BARS @ 12" EPOXY COATED
SHEAR KEY
SHEAR KEY
℄ BLOCKOUT
℄ LEV. BOLT INSERT
WELDED WIRE FABRIC 6X6X¼
POST TENSIONING DUCT FOR 1⅜"⌀ H.S. DEFORMED BAR
SECTION B-B
#3 BARS @ 12" CTS. EPOXY COATED
#4 BARS @ 4" CTS.
2'-0" EMBEDMENT
SECTION C-C
DECK MODULE SPACING
13 @ 8'-0" = 112'-0"
4'-0" BLOCKOUTS (TYP.)
FILL WITH NON-SHRINK CEMENT
℄ BRIDGE
5 STRINGERS @ 15'-4"
71'-6"
A
B
C
D
E
PLAN OF BRIDGE DECK
INITIAL PRESTRESSING FORCE = 180 KIPS PER FOOT WIDTH OF MODULE
POST-TENSIONING FORCE SHALL BE 170 KIPS PER 1-3/8" HIGH STRENGTH BAR
14"
13"
FILL WITH NON-SHRINK GROUT
¾"⌀ STUDS
SECTION D-D
6"
5"
SECTION E-E
SHEAR STUD BLOCKOUT DETAIL
FILL WITH NON-SHRINK GROUT
COAT WITH POURABLE EPOXY
SHEAR KEY
⅞"⌀ LEV. BOLT
2½" BLOCKOUT
LEVELING BOLT INSERT
2½" BLOCKOUT
⅝"⌀ STUD
℄ 6X6X½
DETAIL OF LIFTING HOOK
GENERAL NOTES:
DESIGN SPECIFICATIONS: A.A.S.H.T.O. STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES-LATEST EDITION
DESIGN LOADING: HS 20-44.
DESIGN LOADING INCLUDES 25 P.S.F. ALLOWANCE FOR FUTURE OVERLAY.
PRESTRESSING STRAND SHALL CONFORM TO ASTM-A-416-80 GRADE 270.
CONCRETE: F'C = 4000 PSI F'CI = 3500 PSI
REINFORCING STEEL: ASTM A615 GRADE 40.
MINIMUM CONCRETE COVER SHALL BE AS FOLLOWS
TOP OF SLAB-2" BOTTOM OF SLAB-1" OTHER LOCATIONS-1½"
POST TENSIONING ANCHORAGE ASSEMBLIES SHALL BE CAPABLE OF DEVELOPING THE MINIMUM GUARANTEED ULTIMATE STRENGTH OF THE STRANDS.
SHEAR STUDS: ASTM A108.
BOLTS: ASTM A307.
POST-TENSIONING BARS SHALL BE HIGH STRENGTH PREFORMED BARS CONFORMING TO ASTM A722 AND A29 WITH A FPU = 160 KSI
ERECTION PROCEDURE
1. PLACE NEOPRENE O-RING TUBULAR SIDE FORM MATERIAL ON EDGES OF GIRDER TOP FLANGES. USE EPOXY GLUE TO ATTACH TO GIRDER FLANGES.
2. PLACE FIRST SLAB IN POSITION USING LEVELING BOLTS TO PROVIDE TEMPORARY SUPPORT AT GIRDER LOCATIONS. BOTH SIDES OF BRIDGE MAY BE CONSTRUCTED SIMULTANEOUSLY.
3. TIGHTEN OR LOOSEN LEVELING BOLTS AS NECESSARY TO ACHIEVE PROPER ELEVATION OF SLAB.
4. BRUSH A SUITABLE POURABLE EPOXY MATERIAL ONTO THE BOTTOM HALF OF THE KEY JOINT TO A THICKNESS OF 1/16-INCH.
5. PLACE ADJACENT SLAB IN A SIMILAR MANNER BEFORE THE EPOXY IN THE SHEAR KEY HAS BECOME HARD. PULL SLABS TOGETHER SUFFICIENTLY TO PLACE SHEAR KEYS IN CONTACT. ALIGN POST-TENSIONING DUCTS.
6. PLACE REMAINING SLABS IN SEQUENCE USING SAME PROCEDURE.
7. AUTOMATIC END WELD SHEAR CONNECTORS IN POSITION THROUGH POCKETS PROVIDED IN SLABS.
8. INSTALL POST-TENSIONING STRANDS THROUGH DECK. POST-TENSION CABLES IN SUCH A MANNER THAT THE FORCES WILL BE EQUALLY DISTRIBUTED THROUGH THE SLAB.
9. THROUGH POCKETS PROVIDED IN SLAB, PLACE NON-SHRINK GROUT TO FILL VOID BETWEEN SLAB AND GIRDER, AS WELL AS POCKET. GROUT SHALL BE PLACED ON EACH GIRDER BEGINNING AT THE NEAR END AND PROCEEDING TO THE FAR END OF EACH SLAB SEQUENTIALLY.
10. GROUT SHALL NOT BE PLACED IN A NEW POCKET UNTIL THE GROUT FROM THE ADJACENT POUR LOCATION HAS BECOME VISIBLE IN THE NEW POCKET.
11. FORM AND PLACE CONCRETE IN CENTER SECTION OF DECK, AND FOR PARAPETS.
12. INSTALL BRIDGE RAILS.
SI CONVERSIONS
1 FOOT = .3048 METERS
1 KIP = 4448 NEWTONS
1 KSI = 6.895 MPa
1 PSF = 47.88 N/m²
PREPARED BY
BYRD, TALLAMY, MacDONALD and LEWIS
FHWA PROJECT DOT-FH-11-9519
APPLICATIONS OF TRANSVERSE PRESTRESSING FOR BRIDGE DECKS
CONCEPT NO. 4
NEW BRIDGE
PRECAST HALF WIDTH TRANSVERSELY PRESTRESSED-LONGITUDINALLY POST-TENSIONED BRIDGE DECK

Figure 28. Concept No. 4 - New Bridge

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Modular Deck

The modular precast prestressed deck slab is placed in two sections, each comprising approximately one-half the structure width. The area remaining between the individual modules is filled with cast-in-place concrete to complete the bridge deck width. The modules are 8 ft. wide and the individual units weigh about 17 tons (15,200 kg).

Mild reinforcement is provided in the slabs for distribution and for temperature and shrinkage. The bottom mild reinforcement consists of a welded wire fabric mesh. The top longitudinal reinforcement is #4 bars, epoxy-coated.

Longitudinal post-tensioning ducts are provided at intervals between the two levels of prestressing wire for longitudinal post-tensioning of the deck.

Slab Design - In the preliminary design stage, the overhang dimension and subsequent girder spacing were varied to maximize the efficiency of the prestressing, equating the maximum cantilever moment to the corresponding interior moment as determined in accordance with AASHTO specifications. This resulted in a 15'-4" (4.7 m) beam spacing with a 5'-1" (1.5 m) deck cantilever.

The minimum slab thickness based upon the AASHTO formula is .82 ft. (.25 m). A thickness of 10 in. (254 mm) is used. The prestressing strands are placed in two rows, 4 in. (102 mm) apart vertically, placed concentric with the centerline of the slab depth. This provides a minimum clear cover over the prestressing strands of 2-3/4 in. (70 mm).

Bending moments are determined by using formulas developed for main reinforcement perpendicular to traffic. Because the slab is continuous over more than 3 supports for live load a continuity factor of 0.8 is used. For purposes of this concept an allowance of 25 psf (122 Kg/m^2) is provided for application of a future wearing surface. Design calculations are based on standard AASHTO HS 20-44 truck loads.

The slab is designed for no tension under live load, eliminating the need for mild reinforcement in the precompressed tensile area.

The determination of the prestressing force required is obtained by equating the precompression stress level to the maximum fiber stress produced by application of all design loads. Prestress losses are estimated at 20 percent in accordance with AASHTO specifications as in the other concepts.

The initial prestress force required is calculated using the formula given previously. In this example, a total of 30, 1/2" Ø (13 mm) strands of 270 KSI (1861.58 MPa) ultimate strength are required. These are placed in two rows of 15 each.

The average development length for a 1/2" Ø (13 mm) prestressing strand is 41 in.(1041 mm). This is less than the distance to the point of maximum cantilever moment, which is 56 in.(1422 mm). The development length is therefore adequate to transfer the full prestressing force into the deck module before the point of maximum moment is reached.

The longitudinal post-tensioning force is applied symmetrically across the deck cross-section. A total of 12 1-3/8" Ø (35 mm) high strength deformed bars are required for the total bridge cross-section.

Compressive strength of concrete at 28 days is determined assuming that the openings provided for shear connectors and other inserts are subtracted from the area available to resist the initial prestressing force. Because these openings are filled only after all prestressing forces have been applied, the initial precompression of the remaining concrete cross-section is increased in proportion to the percentage of the total section loss. This increases the final concrete stress from 3250 to 4000 psi (22.24 to 27.58 MPa).

The deck slab is made continuous for live load and impact forces by providing a mild reinforced concrete section between adjacent slabs at the bridge centerline. The moment computed for live load and impact forces for the precast slab is used to calculate the steel required at this location. This results in 1/2" Ø (13 mm) grade 60 bars at 4 in. (102 mm) center to center. The concrete stress is 1000 psi (6.89 MPa). Complete calculations are provided in Appendix A.

Deck Detail - The transverse joints between slabs are a double female type (see the shear key detail on Figure 28 for dimensions). This joint configuration was selected to provide a simple yet tight joint. This joint is developed in a manner similar to that in Design Concept Number 2.

Leveling bolts are provided for vertical adjustment and temporary support for the slab units. Once the first slab has been placed and aligned vertically by adjusting the projection of the bolt end below the bottom of the slab, additional slabs can be aligned in the same manner.

Bearing Requirements - The area between the top of the support girder and the bottom of the precast deck modules must be filled with a non-compressible material capable of transferring the vertical loads from the weight of the deck and the applied live loads into the support girder. Tubular sponge neoprene is provided in a manner similar to that for the first three design concepts.

The modular deck will be attached to the wide flange beams by use of shear studs welded to the top flange through preformed 6 in. x 14 in. (152 x 356 mm) cutouts in the deck module. Figure 28 in Section D-D shows the configuration of these cutouts and the shear studs.

Construction Sequence - The construction sequence for placement of the modular deck system is as follows:

1. Place neoprene O-ring tubular side form material on the edges of the top flange of the support girders using an epoxy glue.
2. Place the first deck module in position at the end of the bridge, using the leveling bolts to raise or lower the slab to its proper elevation. These leveling bolts can be turned down before placement so that the initial elevation of the module is above its final anticipated elevation. Shim pads can then be placed beneath the edge of the module at center beam location, and the leveling bolts used to drop the module to its correct elevation while at the same time providing proper temporary bearing under each deck module.
3. Place the first deck module for the remaining half of the bridge.

4. Brush a suitable pourable epoxy mortar on the lower leading edge of the module just positioned.

5. While the epoxy mortar is still in a plastic state, place the second deck module as provided in step 2 above. The new slab must be aligned both vertically and horizontally. The post-tensioning ducts must be properly aligned and the bottom of the joint must be tight to within a tolerance of 1/16 in. (1.6 mm) to guarantee bonding between the concrete and the epoxy mortar.

6. Fill transverse shear joint with suitable non-shrink grout, striking top off flush with the top surface of slab.

7. Place remaining slabs in sequence using same procedure.

8. Weld shear connectors in position through preformed pockets provided in the individual deck modules during fabrication.

9. Place post-tensioning bars through post-tensioning ducts provided. Use hydraulic jacks, post-tension each bar to initial prestress force of 89 tons (81 t). The bars should be stressed in symmetrical manner about the centerline of the deck module. The post-tensioning should be accomplished in two steps. The first step should place an initial tension on the post-tensioning bars of approximately 50 tons (45.5 t). The final jacking sequence should increase the tension to the final value of 89 tons (81 t).

10. Using the preformed pockets in the deck module, place non-shrink grout in the space between the deck module and the support girder. Care must be exercised to assure that this cavity is completely filled with grout. The placement of grout should commence from one end of the slab forward. Grout should not be placed in an adjacent preformed pocket until grout from the adjacent pour is visible in the new pocket.

11. Place concrete in center section of deck over center support.

CHAPTER V. COST BENEFIT ANALYSIS

FACTORS CONSIDERED

In developing a cost benefit analysis for the modular precast deck system compared with the conventional cast-in-place deck system, several factors must be considered. Elements involved in the systems must be quantified and cost developed for each. Other factors must also be considered and, where possible, related to overall cost. These include the time required to construct the facility, scheduling impact and maintenance of traffic during construction. There are other items that should be considered that are difficult to quantify such as inconvenience to motorist, traffic congestion with resulting delays on detour routes, and the like.

Initial Construction Cost

For each of the four concepts developed in Chapter IV, an estimate was made for initial cost using the precast modular system and using conventional cast-in-place construction. Items included in the analysis are deck concrete and reinforcing required, miscellaneous joint material, plus labor and equipment required to construct the deck. Estimated cost for these items was obtained from the Dodge Manual, the Means Cost Guide and from actual project bid data in the Virginia and Pennsylvania areas.

Production rates for various construction activities required in cast-in-place construction were obtained from the same sources and are summarized in Table 1. Crew size for each activity is also given in this table.

Table 2 summarizes how the handling of materials varies between pre-casting and cast-in-place construction. These differences must be considered in the development of construction cost estimates for each system.

Differences in components other than the concrete deck which could occur as a result of changes in dead load between the two alternates were not included in the analysis. In all cases, these would support the use of the precast modular system if they had been included.

Table 1. Production Rates for Cast-in-Place Construction Activities

Activity	Production Rate & Required Crew
Forming	Placing wooden forms -- Crew - 1 foreman 4 carpenters 1 laborer Production - 13 sq.ft./man hr. Placing steel forms -- Crew - 1 foreman 2 carpenters 2 iron workers 1 compressor operator 1 hoisting engineer Production - 25 sq.ft./man hr.
Placing Reinforcement	Crew - 2 ironworkers Production - 200 lbs./man hr.
Placing Concrete	Crew - 1 foreman 3 laborers 1 hoisting engineer 1 compressor operator Production - 1.88 c.y./man hr.
Finishing Concrete	Crew - 1 foreman 2 laborers 2 finishers Production - 87.5 sq.ft./man hr.
Curing Concrete	Crew - 1 foreman 3 laborers 1 teamster Production - 17.4 sq. yd./man hr.
Removing Forms	Wooden forms - stripping, cleaning and moving Crew - 1 foreman 4 carpenters 1 laborer Production 31 sq.ft./man hr.

Table 2. Comparison of Materials Handling Used in Precast Concrete Construction and Cast-in-Place Construction

MATERIAL	CAST-IN-PLACE	PRECAST
Concrete	Brought to bridge site by Ready-mix trucks from batch plant. Plant may be long distance from site. Access to site can be difficult. Ready-mix plant provides concrete for many different applications. Must change mix design for customer.	Mix produced at plant site. Minimal transportation requirements. Mix specially formulated for precast applications.
Reinforcing Steel	Transported to individual bridge site. Generally cut and bent to length and shape by subcontractor prior to shipment to site. Steel stored on site, wherever space is available.	Transportation to single plant location instead of numerous sites. Plant cuts and bends as required. No need for subcontractor. Steel is stored in centrally located area.
Forms	Wooden forms must be built to fit and supported during placing of concrete and subsequent curing. Steel forms, if used, must be transported to bridge site and welded to beam flange. Overhang forms must be built and supported. Forms must be accurately placed to grade in order to assure proper alignment.	Forms are either permanent or semi-permanent and fully reusable.

On-Site Construction Time

There is a significant reduction in the time required to erect the bridge deck when using precast modular units since the major portion of the forming and casting is done at the factory. The estimated rate of erection for the precast panels is eight units per day, or approximately 600 sq.ft. (55.8 m). This could vary depending on the size and weight of each unit. Miscellaneous joint filling and the like will reduce the overall rate for deck construction to approximately 400 sq.ft. (37.2 sq. m) per day. Cast-in-place construction is estimated at a rate of approximately 100 sq.ft. (9.3 sq. m) per day including curing and form stripping.

Scheduling

Production scheduling is essential to the timely and efficient execution of a construction project. Removal of elements from the critical path greatly eases the problem of coordination, as well as reducing the overall time required to complete the project. Precast construction effectively accomplishes this goal. The fabricator is able to schedule his workload to his greatest advantage. Fabricators generally have many different projects in their plant at any one time permitting manpower and equipment to be moved from project to project as needed. In this way, small projects can be handled in an efficient and economical manner. With full utilization of manpower, equipment and facilities, indirect costs including overhead and depreciation are greatly reduced, resulting in a more economical end product.

Traffic Control

When an existing structure is put out of service, either completely by closure, or partially by diversion of traffic, some form of traffic control is required. In the case of a bridge closing, the on-site traffic control is limited to signs and barricades preventing access to the structure. For any structure closure to be effective, proper alternative routes must be provided. For this, additional signs are needed to detour traffic to other suitable crossings. In rural areas, such detours may require a few signs while in urban areas, wholesale rerouting of traffic may be necessary, with several crossings recommended to prevent a single alternative route from being overtaxed by a sudden influx of vehicles.

Permitting traffic to continue to use a portion of a structure while the other portion is under construction is a costly and potentially dangerous practice. Vehicles and construction crews are continually kept in close proximity to one another. Traffic lane widths are often decreased below minimum standards. Shoulder areas are often eliminated. Portable barriers are all that separate the flow of traffic from the construction area. For these reasons, maintenance of traffic through the work zone should be kept to a minimum.

Each case needs to be evaluated independently to properly estimate the cost for maintaining traffic through the worksite or maintaing a detour route. For this study, reasonable assumptions for these costs are included with the analysis for each concept.

ANALYSIS

A detailed cost benefit analysis for each of the four design concepts developed in Chapter IV is included in Appendix B. A summary of the results of this analysis for each concept follows:

Design Concept No. 1

The total material and labor cost, excluding traffic control requirements for the precast system is \$12.91 per sq.ft. (\$138.78 per sq. m) while the cost for the conventional deck is \$10.28 per sq.ft. (\$110.50 per sq. m), resulting in a 26% higher cost for the precast modular deck system.

When the cost of traffic control is added, the modular precast system is approximately 2% less costly.

It is significant that the on-site construction time for placing the precast deck modules is only 7 days, while for the conventional system the time is 41 days. This creates not only a significant cost item, but reduces the inconvenience to the traveling public.

Design Concept No. 2

This concept illustrates the use of a longitudinal system of prestressing. The support configuration is such that for conventional construction, a slab 13 in. (330 mm) in thickness is required to comply with normal design parameters using standard strength concrete and mild reinforcement. Although the additional 3-1/2 in. (89 mm) of concrete depth will require strengthening of the floor beams and truss members, the resulting additional cost is not considered in the cost analysis.

The analysis indicates that the precast method of construction is approximately 12% less costly, primarily due to the decreased thickness of the slab.

It is significant that the on-site construction time for placing the precast deck modules is 5 days, while for the conventional system it is 25 days.

In this cost comparison, no actual determination is made relating to the cost of inconvenience. Were this done, the benefit of the precast system would be further increased.

Design Concept No. 3

Design Concept No. 3 is an application developed for new bridge construction using composite action between the slab and support system. While the support structure would have to be strengthened to support the heavier deck required for cast-in-place construction this was not considered in the cost analysis.

The analysis indicates that the precast method of construction is approximately 28% less costly. In this concept, most of the cost savings is in the decreased deck slab thickness.

Design Concept No. 4

This concept illustrates the use of a transversely prestressed precast deck on a support configuration that, for conventional construction, would require a slab 12-1/2 in. (318 mm) in thickness based on normal design parameters and standard strength concrete and mild reinforcement.

Although the additional 3 in. (76 mm) of concrete depth will necessitate an increase in the load carrying capacity of the support system, this is not considered in this cost analysis.

The analysis indicates that the precast method of construction is approximately 16% less costly than the cast-in-place alternative based on initial costs.

COST BENEFIT SUMMARY

In each of the concepts developed for new bridge construction, the precast, prestressed modular deck system proved to be the most economical when considering initial cost only. For the two concepts developed for deck replacement, the precast alternative is also more economical when considering initial construction cost and traffic control cost.

Table 3 summarizes the cost analysis for each concept and for each alternative.

Table 3. Summary of Cost Benefit Analysis

CONSTRUCTION	Concept No.	Cast-in-Place			Modular Precast			% Savings	Remarks
		Initial Cost		Field Time for Deck Constr. (days)	Initial Cost		Field Time for Deck Constr. (days)		
		Total $	Unit ($ s.f.)		Total $	Unit ($ s.f.)			
DECK REPLACEMENT	1	64,905	13.75	41	63,747	13.50	7	2 (Modular)	Cost of traffic control -- critical item
	2	37,217	13.89	25	33,227	12.40	5	12 (Modular)	Strenghthening of framing system for conven. construction not included
NEW BRIDGE	3	35,239	14.79	20	27,446	11.52	4	28 (Modular)	Cost of heavier framing for conventional system not included
	4	119,022	14.88	26	102,633	12.83	13	16 (Modular)	Cost of heavier framing for conventional system not included

CHAPTER VI. CONCLUSIONS

This study has shown that the concept of transverse prestressed, precast modular bridge deck construction has definite advantages when compared with conventional cast-in-place deck construction using mild reinforcement. A review of bridges in service with precast decks indicates that these have performed well when compared with cast-in-place construction.

Through the development of four design examples, the concept has been demonstrated to be applicable to both new bridge construction and to replacement deck construction. In each of these cases, a cost estimate was made and in each instance the precast alternative proved to be more economical.

This economy is primarily the result of a decrease in the amount of materials required due to the increased structural efficiency of the prestressed deck. Where time is a factor which has quantifiable costs, this further enhances the economic viability of the concept of precast prestressed construction.

Other benefits have been identified with modular construction on new bridge construction and for deck replacement.

New Bridge Construction

- Improved structural efficiency
- Decreased number of support elements
- Decreased manpower requirements
- Decreased construction time

Replacement Deck Construction

- Decreased on-site construction time and manpower requirements
- Decreased time required for traffic control
- Decreased inconvenience to the traveling public
- Decreased dead load weight and potential for increase in live load capacity.

Also, precast concrete elements fabricated in a controlled environment under "factory" conditions provides for improved quality control which can ultimately result in improved durability of the completed deck.

In the design examples several concepts have been presented which have not been field tested. These include modular panels spacing between floor beams, closure details at the support members, details for developing composite action, joints and others.

Needed research identified by this study includes:

1. The development of improved connections between modular deck slabs and the supporting structure.
2. The evaluation of benefits of two-way prestressing.

This study has shown that the use of transversely prestressed precast modular deck elements for concrete bridges is a viable alternative to conventional cast-in-place bridge construction. There is evidence that it can effect a savings in the cost of construction and reduce impact upon road users by decreasing construction time.

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APPENDIX A

STRUCTURAL COMPUTATIONS

Design Concept Number 1

F.W.S. Load = 30 psf

Wt. of Concrete = 150 pcf

Width of top flange = 16 in.

Effective Span = c.c. Beam Spacing - 1/2 flange width
= (6'-6") - 8" = 5'-10"

$$\text{M.D.L.} = \frac{\left[(150 \times 7/12) + 30\right] \times 5.8}{10} = 395 \text{ ft-lb}$$

$$M_{LL} = \frac{S+2}{32} \times P_{20} = \frac{5.8 + 2}{32} \times 16000 = 3900 \text{ ft-lb} \times .8 = 3120 \text{ ft-lb}$$

$$I = \frac{50}{125+L} = \frac{50}{125+5.8} = .38 \text{ Use .30 Max.}$$

$$M_I = 3120 \times .3 = 936 \text{ ft-lb}$$

$$M_{\text{total}} = 395 + 3120 + 936 = 4451 \text{ ft-lb}$$

Slab and prestressing will be placed parallel to skew, therefore, moments must be increased by value of $1/\cos 12^\circ = 1.02$

Moment on skew = 1.02 x 4451 = 4550 ft-lb per ft

(24810 Nm/m)

Bending Moment on Cantilever

Eff. cant. = 4' -3" - 4" = 3' -11"

$$M_{DL} = (150 \times .58 + 30) \times 1.92^2 + \left[(150 \times 1.33 \times 2.0) + 20\right] \times 2.92$$

$$M_{DL} = 216 + 1223 = 1439 \text{ ft-lb per ft}$$

$$M_{LL} - E = 0.8\,X + 3.75 = 0.8 \times .92 + 3.75 = 4.49$$

$$M_{LL} = \frac{P}{E}\,X = \frac{1600}{4.49} \times .92 = 3280 \text{ ft-lb per ft}$$

$$M_I = .3 \times 3280 = 984 \text{ ft-lb per ft}$$

$$M_{\text{Total}} = 1439 + 3280 + 984 = 5703 \text{ ft-lb per ft}$$

(25890 Nm/m)

Design Concept Number 1

Structural Design of Cantilever Slab Using Prestressing Strands

Slab depth = 7 in.

$I_{slab} = \frac{bh^3}{12} = \frac{12x7^3}{12} = 343 \text{ in.}^4$ Area = bh = 12 x 7 = 84 in.^{2}

$S = \frac{I}{C} = \frac{343}{3.5} = 98 \text{ in.}^3$

Maximum Concrete Stress = $\frac{M}{S} = \frac{5700 \times 12}{98}$ = 698 psi (4.81 MPa)

Assuming no tension under live load, precompression of slab must equal maximum concrete stress.

∴ Max. Precomp. = 698 psi

Assume 20% loss in prestress under service loads

$$\text{Init. Prest.} = \frac{\text{Max. Precom. x area}}{\text{1 - prestress loss}}$$

$$= \frac{698 \times 84}{1 - .2} = 73290 \text{ lb per ft}$$

(1079 KN/m)

Because slabs will be post-tensioned longitudinally, component of this force can be used to decrease the prestressing force required.

P = 250 psi x .8 x sin 12° = 42 psi

Force Reductions = 42 x 84 ÷ 8 = 4410 lb per ft

Design Concept Number 1

P init = 73290 - 4410 = 68880 lb per ft

Use 7/16 in. dia. by 270 KSI prestressing strands.

P init = 21,700 lb per strand (96.7 KN)

$$\text{No. of Strands} = \frac{\text{P init}}{\text{P init/St.}} = \frac{68880}{21700} = 3.17 \text{ strands}$$

Total width = 8 ft   Total Strands = 3.17 x 8 = 25.4

Use 26 Strands. Place in 2 rows, 2 inches apart vertically.

Check for maximum concrete compression.

Blockouts will decrease effective concrete area at supports. Area lost includes 12 in. for shear connectors plus 6 in. for leveling bolts.

Minimum width available for precompression = 8 ft - 1.5 ft = 6.5 ft

Actual P init = 26 strands x 21,700 lb = 564,200 lb.

$$\text{Init. Concrete Precomp.} = \frac{564{,}200}{7 \text{ in.} \times 78 \text{ in.}} = 1033 \text{ psi}$$

Final Concrete Precomp. = 1033 psi x .8 = 827 psi

Maximum DL + LL + I Con. Stress = 698 psi

Maximum Conc. Comp. = 827 + 698 = 1524 psi

$$fc = .4\ f'_c \qquad f'_c = \frac{fc}{.4} = \frac{1524}{.4} = 3800 \text{ psi}$$

Use 4000 psi High Strength Concrete
(27.58 MPa)

Design Concept Number 1

Longitudinal Post-Tensioning

Final Post-tensioning Force Desired = 200 psi

Use 20% Loss - Initial Force = $\frac{200}{.8}$ = 250 psi

Overall Deck width = 60'-6"

Total Area of End = 60.5 x .58 = 35.09 ft^2

Initial P = 35.09 x 12^2 x 250 = 1,263,200 lb

Use High Strength Deformed Bars -

Initial Tensioning = 46.2 KIP (205.95 KN)

No. of Bars = $\frac{1263.2}{46.2}$ = 27.3 - Use 28 bars

Use 14 on each half of the structure

Design Concept Number 2

F.W.S. = 25 psf

Weight of concrete = 150 pcf

Width of top flange 16 in.

Effective Span S = (15'-11")-(1'-4") + 8 in. = 15'-3"

Use 9 1/2 in. thick slab

D.L. = $\frac{9.5}{12}$ x 150 = 118.75 psf

F.W.S $\quad \underline{25.00}$

143.75 psf

Wheel Load = 16^{k}

Distribution of wheel load E = 4 + 0.06 S = 4.91 ft

Wheel load per ft of width = $\frac{16}{4.91}$ = 3.26^{k}

From AISC Table A3.0

$- M_{DL}$ = 0.1 x 143.75 psf x 15.92^2 = $\underline{\underline{3{,}566 \text{ lb-ft}}}$

$+ M_{DL}$ = $\frac{0.08}{0.1}$ x 3566 lb-ft = $\underline{\underline{2{,}853 \text{ lb-ft}}}$

Live Load Moments

Wheel Span 1 - M_{LL} = 0.1024 x 3.26 kips x 15.92 ft = 5.190 ft-kips

Wheel Span 2 - M_{LL} = 0.076 x 3.26 kips x 15.92 ft = $\underline{3.994}$

Total - M_{LL} = 9.134

I = $\frac{50}{125+15.92}$ = 0.36 Use 0.3 $\quad \underline{\text{x } 1.3}$

$- M_{LL}$ + I $\quad$ 11.87 ft-kips

M_{DL} + LL + I = 3,566 ft-lb + 11,870 ft-lb = 15,436 ft-lb (68.63 kNm/m)

Slab d = 9 1/2 in.

I = $\frac{bd^3}{12} = \frac{12 \times 9.5^3}{12}$ = 857 in.^{4} A = 9.5 in. x 12 in. = 114 in.^{2}

S = $\frac{857}{4.75}$ = 180.5 in.^{3}

Design Concept Number 2

Final Concrete Stress to Resist Moment

$$\frac{15436 \text{ ft-lb} \times 12}{180.5 \text{ in.}^3} = 1026 \text{ psi } (7.07 \text{ MPa})$$

Precompression of slab must equal maximum concrete stress

Stress must be doubled Max. = 1026 psi

Min Stress = 0

Assume 20% loss $\frac{1026 \text{ psi} \times 114 \text{ in.}^2}{0.8} = 146235 \text{ lb}$

No. of strands = $\frac{146235}{28900} = 5.06$ strands/ft

6'-10 1/2" wide slab. Strands req'd = 5.06 x 6.875 = 34.8

Use 36 strands. Place in 3 rows, 2 inches apart vertically.

Check for maximum concrete compression.

Blockouts will decrease effective concrete area at supports. Area lost includes 12 inches for shear connectors plus 6 inches for leveling bolts.

Minimum width available for precompression = 6.875 - 1.5 = 5.375 ft.

Actual P int = 36 x 28900 lb = 1040400 lb.

Initial concrete precompression = $\frac{1040400}{9.5 \times 64.5} = 1698$ psi.

Final concrete precompression = 1698 x .8 = 1358 psi.

Max. DL + LL + I Concrete stress = 1026 psi.

Max. Conc. Comp. = 1358 + 1026 = 2384 psi.

$f_c = .4 f'_c \qquad f'_c = \frac{f_c}{.4} = \times \frac{2384}{.4} = 5960$ psi.

Use 6000 psi High Strength Concrete.

(41.37 MPa)

Design Concept Number 3

F.W.S. Load = 25 psf

Wt. of concrete = 150 pcf

Width of top flange = 16 in.

Effective Span = c.c. Beam Spacing - flange width
= (9'-6") - 16 in = 8'-2"

$$M.D.L. = \frac{[(150 \times 7.5) + 25] \times 8.17}{10} = 793 \text{ ft-lb}$$

$$M_{LL} = \frac{S + 2}{32} \times P_{20} = \frac{8.17 + 2}{32} \times 16000 = 5085 \text{ ft-lb} \times .8 = 4068 \text{ ft-lb}$$

$$I = \frac{50}{125+L} = \frac{50}{125 + 8.17} = .38 - \text{Use } .30 \text{ Max.}$$

$$M_I = 4068 \times .3 = 1200 \text{ ft-lb}$$

$$M_{Total} = 793 + 4068 + 1220 = 6081 \text{ ft-lb}$$

Slab and prestressing will be palced parallel to skew. Therefore, moments must be increased by value of 1/cos 20^{0} = 1.06

Moment on skew = 1.06 x 6081 = 6470 ft-lb per ft

(28511 Nm/m)

Bending Moment on Cantilever

Eff. Cant. = 4'1" - 8" = 3'5"

$$M_{DL} = \frac{[(150 \times .63) + 25] \times 2.08^2}{2} + [(150 \times 1.38 \times 1.33) + 20] \times 2.75$$

$$M_{DL} = 257 + 812 = 1069 \text{ ft-lb per ft.}$$

$$M_{LL} \quad E = 0.8X + 3.75 = 0.8 \times 1.08 + 3.75 = 4.61$$

$$M_{LL} = \frac{P}{E}X = \frac{16000}{4.61} \times 1.08 = 3748 \text{ ft-lb per ft.}$$

$$M_I = .3 \times 3748 = 1125 \text{ ft-lb per ft.}$$

$$M_{Total} = 1069 + 3748 + 1125 = 5942 \text{ ft-lb per ft.}$$

(26980 Nm/m)

Design Concept Number 3

Structural Design of Slab Using Prestressing Strands

Slab depth = 7 1/2 in.

$I_{slab} = \frac{bh^3}{12} = \frac{12 \times 7.5^3}{12} = 422 \text{ in.}^4$ $A = bh = 12 \times 7.5 = 90 \text{ in.}^2$

$S = \frac{I}{C} = \frac{422}{3.75} = 112.5 \text{ in.}^3$

Maximum Concrete Stress $= \frac{M}{S} = \frac{6470 \times 12}{112.5} = 690$ psi

Assuming no tension under live load, precompression of slab must equal maximum concrete stress.

$\therefore$ Max. Precomp. = 690 psi (4.76 MPa)

Assume 20% loss in prestress under service loads.

Init. Prest. $= \frac{\text{Maximum Precomp. x area}}{\text{1 - Prestress Loss}}$

$= \frac{690 \times 90}{1 - .2} = 77625$ lb. per ft.

(1125.5 KN/m)

P init = 77625

Use 7/16 in. diameter - 270^{ksi} prestressing strands

P init = 21,700 lb per strand (96.7 KN)

No. of Strands $= \frac{\text{P init}}{\text{P init/st}} = \frac{77625}{21700} = 3.58$ strands

Total Width = 8'6-1/2" Total Strands = 3.58 x 8.54 = 30.55

Use 32 Strands. Place in two rows, 2-1/2 inches apart vertically.

Design Concept Number 3

Check for Max. concrete compression. Blockouts will decrease effective concrete area at supports. Area lost includes 28" for shear connectors plus 6" for leveling bolts.

Min. Width available for precompression = (8.54) - 2.33 - .5 = 5.71 ft.

Actual P init = 32 Strands x 21,700 lb. = 694400

Init. Concrete Precomp. = $\frac{694400}{7.5 \times 68.5}$ = 1352 psi

Final concrete precomp. = 1352 psi x .8 = 1081 psi

Max. DL + LL + I Constress = 690 psi

Max. Conc. Comp. = 1081 + 690 = 1771 psi

$f_c = .4f'_c \qquad f'_c = \frac{fc}{.4} = \frac{1771}{.4} = 4427$ psi

Use 4500 psi High Strength Concrete

(31.03 MPa)

Design Concept Number 4

F.W.S. Load = 25 psf

Wt. of concrete = 150 pcf

Width of top flange = 16"

Effective Span = c.c. Beam Spacing - 1/2 flange

Width = (15'4") - 8" = 14'8"

$$\text{M.D.L.} = \frac{[(150 \times 10/12) + 25] \times 14.67^2}{10} = 3228 \text{ ft-lb}$$

$$M_{LL} = \frac{S + 2}{32} \times P_{20} = \frac{14.67 + 2}{32} \times 16000 = 8335 \text{ ft-lb.} \times .8 = 6668 \text{ ft-lb}$$

$$I = \frac{50}{125 + L} = 125 + \frac{50}{125 + 14.67} = .36 \quad \text{Use .30 Max.}$$

$$M_I = 6668 \times .3 = 2000 \text{ ft.-lb.}$$

$$M_{Total} = 3228 + 6668 + 2000 = 11896 \text{ ft.-lb.}$$
$$= (16130 \text{ Nm})$$

Bending Moment on Cantilever

Eff. Cant. = 5'1" - 4" = 4'9"

$$M_{DL} = \frac{[(150 \times .83) + 25] \times 3.42^2}{2} + [(150 \times 1.58 \times 1.33) + 20] \times 4.08$$

$$M_{DL} = 874 + 1368 = 2242 \text{ ft-lb per ft.}$$

$$M_{LL} \quad E = 0.8X + 3.75 = 0.8 \times 2.42 + 3.75 = 5.69$$

$$M_{LL} = \frac{P}{E}X = \frac{16000}{5.69} \times 2.42 = 6805 \text{ ft-lb per ft.}$$

$$M_I = .3 \times 6805 = 2041 \text{ ft-lb per ft.}$$

Mom. Total = 2242 + 6805 + 2041 = 11088 ft-lb per ft.
= (15035 Nm/m)

Design Concept Number 4

Structural Design of Slab Using Prestressing Strands

slab depth = 10 in.

$$I_{slab} = \frac{bh^3}{12} = \frac{12 \times 10^3}{12} = 1000 \text{ in.}^4 \quad A=bh=12\times10=120 \text{ in.}^2$$

$$S = \frac{I}{C} = \frac{1000}{5} = 200 \text{ in.}^3$$

$$\text{Maximum Concrete Stress} = \frac{M}{S} = \frac{11896}{200 \text{ in.}^3} = 714 \text{ psi} \quad (4.92 \text{ MPa})$$

Assuming no tension under live load, precompression of slab must equal maximum concrete stress.

$\therefore$ Max. Precomp. = 714 psi.

Assuming 20% loss in prestress under service loads

$$\text{Init Prest.} = \frac{\text{Max. Precomp. x Area}}{\text{1 - Prestress loss}}$$

$$= \frac{714 \times 120}{1 - .2} = 107060 \text{ lb-ft}$$

P init = 107060 lb per ft

Use 1/2 in. dia - 270 kSI prestressing strands

P init = 28,400 lb per strand

$$\text{No. of Strands} = \frac{\text{P init}}{\text{P init/St}} = \frac{107060}{28400} = 3.77 \text{ strands}$$

Total width = 8 ft $\therefore$ Total Strands = 3.77 x 8 = 30.16

Use 30 strands. Place in 2 rows, 4 inches apart vertically.

Check for Max. concrete compression. Blockouts will decrease effective concrete area at supports. Area lost includes 12 in. for shear connectors plus 6 in. for leveling bolts. Min. width available for precompression = 8 ft-1.5 ft.= 6.5 ft.

Design Concept Number 4

Actual P init = 30 strands x 28400 lb = 852000 lb

Init. Concrete Precomp. = $\frac{852000}{10 \text{ in. } \times 78 \text{ in.}}$ = 1092 psi

Final concrete Precomp. = 1092 psi x .8 = 874 psi

Max. DL. + LL. + I con. stress = 714 psi

Max. Conc. Comp. = 874 + 714 = 1588 psi

$fc = .4f'_c \therefore f'_c = \frac{f_c}{.4} = \frac{1588}{.4} = 3970$ psi

Use 4000 psi High Strength Concrete.
(27.58 MPa)

Longitudinal Post-Tensioning

Final Post-tensioning force desired = 200 psi

Use 20% loss - Initial Force = $\frac{200}{.8}$ = 250 psi

Overall Deck Width = 71.5 ft.

Total area of end = 71.5 x .83 = 59.35 ft^2

Initial P = 59.35 x 12^2 x 250 = 2,136,420 lb

Use High Strength Deformed Bars-Try 1 3/8 in. Ø

Initial Tensioning = 187.2^k

No. of Bars = $\frac{2136.4}{187.2}$ = 11.4 - Use 12 Bars

Use 6 on each half of the structure

Init. Prestension = $\frac{2136.4}{12}$ = 178 Kips
(791.7 KN)

Design of center support for live load

L.L. + I = 6668 + 2000 = 8668 ft-lb per ft

Effective "d" = 10 - 1.5 - .25 = 8.25 in.

$A_s = \frac{M}{fsjd} = \frac{8668 \times 12}{2400 \times .88 \times 8.25}$ = .60 $in.^2$ per ft x 8 ft = 4.80 $in.^2$

Use 24 - # 4 @ 4 in. c.c. A_s = 24 x .2 = 4.8 $in.^2$

APPENDIX B

COST BENEFIT ANALYSIS

Design Concept 1 - Bridge Rehabilitation

Transversely Prestressed Deck Modules

COST ESTIMATE

Deck Module

Price per sq.ft.

Assume 6 units per bed

8' wide 30'3" long

242 sq.ft. - 5.23 c.y./unit

4000 psi concrete

Item		Cost/sq.ft.
Concrete (4000 psi) $33 x 5.23 ÷ 242	=	$ .71
Strand - 26 strands x 35' long x $0.21/ft ÷ 242	=	.79
Reinf. Steel - 1 lb x $0.20lb	=	.20
Embedded Steel - $100/unit ÷ 242	=	.41
Misc.	=	.10
Post. Tens. - $0.80/ft. x 112' ÷ 242	=	.37
Material	=	$ 2.58
On-Line Labor - 8 men x 8 hrs x $8 (avg) ÷ (6 x 242)	=	$ .35
Off-Line Labor (est)	=	.12
Labor Overhead @ 250%	=	1.18
Labor	=	$ 1.65
Forms - 200 L.F. @ $125 = 25,000 x .50 ÷ 4700	=	$ 2.66
Curing & Misc. Equip = 10,000 x .50 ÷ 4700	=	1.06
Handling Equip. = $250/Day ÷ (6x 242)	=	.17
Equipment	=	$ 3.89
SUB TOTAL	=	$ 8.12
+35% O. H. & Profit	=	$ 2.84
Haul - Truck & Driver at $20/hr, 6 per day 20 x 8 ÷ (6 x 242)	=	$ .11
Crane - $300 5 man crew $13/hr x 8 x 5 = $520 set 8 per day = 820 ÷ (8 x 242)	=	.42
TOTAL COST-IN PLACE	=	$11.35/sq.ft.

Miscellaneous Items

Neoprene O-Ring (installed)	1400 L.F. x $0.40	=	$ 560.00
Shear Connectors	540 units x $1.25	=	675.00
Epoxy Mortar	408 sq.ft.x $0.79	=	285.00
Non-Shrink Grout	820 sq.ft.x $0.40	=	328.00
Post-tensioning Complete	2184 L.F. x $2.25	=	4914.00
Traffic Control - 2 days	7 x $400	=	2800.00
	Total Cost Misc. Items	=	$ 9562.00

Summary

Deck Modules	4720 sq.ft. @ $11.48	=	$54,186.00
Miscellaneous Items		=	$ 9,562.00
	Total Cost	=	$63,747.00

Unit cost = $63,747 ÷ 4720 sq.ft. = $13.50/sq.ft.
= $145.12/sq.m.

ESTIMATED FIELD CONSTRUCTION TIME

Erect Panels	4720 sq.ft. ÷ 242 sq.ft./panel	=	20 panels
	20 panels ÷ 8 panels/day	=	2.5 days

Miscellaneous Work

Preparatory work (est)	=	2.0 days
Grouting and Post-tensioning (est)	=	2.5 days
Total Field Time	=	7.0 days

Conventional Cast-In-Place Deck Slab

COST ESTIMATE

Concrete (3000 psi) - 117 c.y. @ $240	=	$ 28,080.00
Reinforcing bars - Uncoated - 16000 Lb @ $0.46	=	7,360.00
Reinforcing bars - Epoxy Coated - 14200 Lb @ $0.70	=	9,940.00
Shear Connectors - 2500 units @ $1.25	=	3,125.00
Traffic Control - 41 days @ $400	=	16,400.00
TOTAL COST-IN PLACE	=	$ 64,905.00

Unit Cost = $64,905.00 ÷ 4720 sq.ft. = $13.75/sq.ft.
= $147.81/sq.m.

TIME AND MANPOWER ESTIMATE

Formwork - 4300 sq.ft. : 13 sq.ft./mh = 331 mh
6 man crew - 8 hrs/day = 331 ÷ 6 ÷ 8 = 7 days

Placing and Tying Reinforcement 30,200 Lbs ÷ 200 Lbs/mh= 151 mh
2 man crew - 8 hr/day = 151 ÷ 2 ÷ 8 = 9.4 days
Use 4 man crew for 5 days

Concrete placement 117 c.y. ÷ 1.88 c.y/mh = 62.2 mh
6 man crew - 8 hrs/day = 62.2 ÷ 6 ÷ 8 = 1.3 man days

Concrete finishing - 4720 sq.ft. ÷ 87.5 sq.ft./mh = 53.94 mh
4 man crew - 8 hrs/day 53.94 ÷ 4 ÷ 8 = 1.7 days

Concrete Curing - 524 sq.yd. ÷ 17.5 sq.yd./mh = 30 mh
4 man crew - 8 hrs/day 30 ÷ 4 ÷ 8 = .9 man days

Form Removal - 4300 sq.ft. ÷ 31 sq.ft./mh = 139 mh
6 man crew - 8 hrs/day 139 ÷ 6 ÷ 8 = 2.9 days

Total curing time - 2 x 14 days = 28 days

Total days required - 46.8

Assume some overlap of work. Total down time = 41 days

Minimum crew required - 6 men

Maximum crew required - 14 men

Design Concept 2 - Bridge Rehabilitation

Longitudinally Prestressed Deck Module

COST ESTIMATE

Deck Module

Price per sq.ft.

Assume 4 units per bed
6'-10 1/2" wide, 48-6" long
333 sq.ft./unit - 9.78 c.y./unit
6000 psi concrete

Item		Cost/sq.ft.
Concrete (6000 psi) $33 x 9.78 ÷ 333	=	$ .97
Strand - 36 strands x 53' long x $0.28/ft ÷ 333	=	1.60
Reinf. Steel - 1 lb x $0.20/lb	=	.20
Embedded Steel - $100/unit ÷ 333	=	.30
Misc.	=	.10
MATERIAL	=	$3.17
On-Line Labor - 8 men x 8 hrs x $8 (avg) ÷ (4 x 333)	=	$ .38
Off-Line Labor (est)	=	.12
Labor Overhead @ 250%	=	1.25
LABOR	=	$1.75
Forms - 200 L.F. @ $125 = 25,000 x .25 ÷ 2680	=	$2.33
Curing & Misc. Equip. = 10,000 x .25 ÷ 2680	=	.93
Handling Equip. = $250/Day ÷ (6 x 333)	=	.13
EQUIPMENT	=	$3.39
SUB TOTAL	=	$8.31
+35% O.H. & Profit	=	$2.91
Haul - Truck & Driver at $20/hr, 8 per day 20 x 8 ÷ (6 x 333)	=	$ .08
Crane - $300 5 man crew $13/hr x 8 x 5 = $520 set 8 per day = 820 ÷ (8 x 333)	=	$ .31
TOTAL COST-IN PLACE	=	$11.61/sq.ft.

Miscellaneous Items

Neoprene O-Ring (installed)	770 L.F. x $0.40	=	$ 308.00
Shear Connectors	112 units x $1.25	=	140.00
Epoxy Mortar	120 sq.ft. x $0.70	=	84.00
Steel Angles	23 Lb x $0.55	=	1,265.00
Non-Shrink Grout	225 sq.ft. x $1.40	=	315.00
	Total Cost Misc. Items	=	2,112.00

Summary

Deck Modules	2680 sq.ft. @ $11.61	=	$31,115.00
Miscellaneous Items		=	3,112.00
	Total Cost	=	$33,227.00

Unit cost = $33,227.00 ÷ 2680 sq.ft. = $12.40/sq.ft.
= $133.28/sq.m.

ESTIMATED FIELD CONSTRUCTION TIME

Erect Panels	2680 sq.ft./panel	=	8 panels
8 panels ÷ 8 panels/day		=	1.0 day
Miscellaneous Work			
Preparatory work (est)		=	2.0 days
Grouting, etc. (est)		=	2.0 days
	Total Field Time	=	5.0 days

Design Concept 2 - Bridge Rehabilitation

Conventional Cast-In-Place Deck Slab

COST ESTIMATE

Concrete (3000 psi) - 107 c.y. @ \$240 = \$ 25,680.00

Reinforcing bars - Uncoated -8950 Lb @ \$.46 = 4,117.00

Reinforcing bars - Epoxy Coated - 10,600 Lb @ \$.70 = 7,420.00

TOTAL COST-IN PLACE = \$ 37,217.00

Unit Cost = \$37,217.00 : 2680 sq.ft. = \$13.89/sq.ft
= \$149.29/sq.ft.

TIME AND MANPOWER REQUIREMENTS

Formwork - 2500 sq.ft. ÷ 13 sq.ft./mh = 192 mh
6 man crew - 8 hrs/day 192 : 6 ÷ 8 = 4 days

Placing and Tying Reinforcement 19,550 Lbs ÷ 200 Lbs/mh = 98 mh
2 man crew - 8 hr/day = 98 ÷ 2 ÷ 8 = 6.1 days

Concrete placement 107 c.y. ÷ 1.88 c.y./mh = 56.9 mh
6 man crew - 8 hrs/day = 56.9 ÷ 6 ÷ 8 = 1.2 days

Concrete finishing - 2680 sq.ft. ÷ 87.5 sq.ft./mh = 30.6 mh
4 man crew - 8 hrs/day 30.6 ÷ 4 ÷ 8 = .96 days

Concrete Curing - 298 sq.yd. ÷ 17.5 sq.yd./mh = 17 mh
4 man crew - 8 hrs/day 17 ÷ 4 ÷ 8 = .5 days

Form Removal - 2500 sq.ft. ÷ 31 sq.ft./mh = 80.7 mh
6 man crew - 8 hrs/day 80.7 ÷ 6 ÷ 8 = 1.7 days

Total curing time - 14 days

Total days required = 28.5

Assume some overlap of work. Total down time = 25 days

Minimum crew required - 6 men

Maximum crew required - 14 men

Design Concept 3 - New Construction

Transversely Prestressed Deck Modules

COST ESTIMATE

Deck Module

Price per sq.ft.

Assume 6 units per bed
8' wide, 39'0" long
312 sq.ft./unit - 7.22 c.y./unit
4500 psi concrete

Item		Cost/sq.ft.
Concrete (4500 psi) $33 x $7.22 ÷ 312	=	$ .76
Strand 32 strands x 44' long x $0.21/ft ÷ 312	=	.95
Reinf. Steel - 1 lb x $0.20/lb	=	.20
Embedded Steel - $100/unit ÷ 312	=	.32
Misc.	=	.10
MATERIAL	=	$2.33
On-Line Labor - 10 men x 8 hrs x $8 (avg) ÷ (6 x 312)	=	$ .34
Off-Line Labor (est)	=	.12
Labor Overhead @ 250%	=	1.15
LABOR	=	$1.61
Forms - 250 L.F. @ $100 - $25,000 x .25 ÷ 2383	=	$2.62
Curing & Misc. Equip = $10,000 x .25 ÷ 2383	=	1.05
Handling Equip. = $250/Day ÷ (6 x 312)	=	.13
EQUIPMENT	=	$3.80
SUB TOTAL	=	$7.74
+ 35% O.H. & Profit	=	$2.71
Haul - Truck & Driver at $20/hr, 6 per day 20 x 8 : (6 x 312)	=	$ .09
Crane - $300	=	
5 man crew 13/hr x 8 x 5 = $520		
Set 8 per day = 820 ÷ (8 x 312)	=	$ .33
TOTAL COST-IN PLACE	=	$10.87/sq.ft.

Miscellaneous Items

Neoprene O-Ring (installed)	520 L.F. x $0.40	=	$ 208.00
Shear Connectors	768 units x $1.25	=	960.00
Epoxy Mortar	200 sq.ft. x $1.00	=	200.00
Non-Shrink Grout	270 sq.ft. x $0.65	=	175.00
	Total Cost Misc. Items	=	$ 1,543.00

Summary

Deck Modules	2383 sq.ft. @ $10.87	=	$25,903.00
Miscellaneous Items		=	1,543.00
	TOTAL COST	=	$27,446.00

Unit Cost = $27,446.00 ÷ 2382 sq.ft. = $11.52/sq.ft.

= $123.86/sq.m.

ESTIMATED FIELD CONSTRUCTION TIME

Erect Panels	2382 sq.ft. ÷ 312 sq.ft./panel	=	1.0 day
Miscellaneous Work			
Preparatory work (est)		=	1.0 day
Grouting, etc. (est)		=	2.0 days
	Total Field Time	=	4.0 days

Design Concept 3 - New Construction

Conventional Cast-In-Place Deck Slab

COST ESTIMATE

Concrete (3000 psi) - 70 c.y. @ $240 = $16,800.00
Reinforcing bars - Uncoated - 17,030 Lb @ $.46 = 7,834.00
Reinforcing bars - Epoxy Coated - 15,150 Lb @ $.70 = 10,605.00
TOTAL COST-IN PLACE = $35,239.00

Unit Cost = $35,239.00 ÷ 2383 sq.ft. = $14.79/sq.ft.
= $158.95/sq.m.

TIME AND MANPOWER REQUIREMENTS

Formwork - 1700 sq.ft. ÷ 25 sq.ft./mh = 68 mh
7 man crew - 8 hrs/day = 68 : 7 ÷ 8 = 1.2 days

Formwork for overhang - 450 sq.ft. ÷ 13 sq.ft./mh = 34.6 mh
6 man crew - 8 hrs/day = 34.6 ÷ 6 ÷ 8 = .7 days

Placing and Tying Reinforcement 33,000 Lbs ÷ 200 Lbs/mh= 165 mh
2 man crew - 8 hr/day = 165 ÷ 2 ÷ 8 = 10.3 days
Use 4 man crew for 5 days

Concrete placement 70 c.y. ÷ 1.88 c.y./mh = 37.2 mh
6 man crew - 8 hrs/day = 37.2 ÷ 6 ÷ 8 = .8 days

Concrete finishing - 2380 sq.ft. ÷ 87.5 sq.ft./mh = 27.20 mh
4 man crew - 8 hrs/day 27.20 ÷ 4 ÷ 8 = .9 days

Concrete Curing - 264 sq.yd. ÷ 17.5 sq.yd./mh = 15.1 mh
4 man crew - 8 hrs/day 15.1 ÷ 4 ÷ 8 = .5 days

Form Removal - 450 sq.ft. ÷ 31 sq.ft./mh = 14.5 mh
6 man crew - 8 hrs/day 14.5 ÷ 6 ÷ 8 = .3 days

Total curing time - 14 days

Total days required 23.4

Assume some overlap of work. Total down time = 20 days

Minimum crew required - 6 men
Maximum crew required - 14 men

Design Concept 4 - New Construction
Transversely Prestressed Deck Modules

COST ESTIMATE

Deck Module

Price per sq.ft.
Assume 6 units per bed
8' wide, 36'-3" long
290 sq.ft./unit - 8.95 c.y./unit
4500 psi concrete

Item		Cost/sq.ft.
Concrete (4500 psi) $33 x 8.95 ÷ 290	=	$ 1.02
Strand - 30 strands x 40' long x $.28/ft ÷ 290	=	1.16
Reinforced Steel - 1 lb x $.20/lb	=	.20
Embedded Steel - $100/unit ÷ 290	=	.34
Misc.	=	.10
Post. Tens. 48 ft. x $2.50/ft. ÷ 290	=	.41
MATERIAL	=	$ 3.23
On-Line Labor - 10 men x 12 hrs x $8 (avg) ÷ (6 x 290)	=	$ .55
Off-Line Labor (est)	=	.24
Labor Overhead @250%	=	1.98
LABOR		$ 2.77
Forms - 200 L.F. @ $125 = 25,000 x .50 ÷ 8000	=	$ 1.56
Curing & Misc. Equip = 10,000 x .50 ÷ 8000	=	.63
Handling Equip. = $250/Day ÷ (6 x 290)	=	.14
EQUIPMENT	=	$ 2.33
SUB TOTAL		$ 8.33
+ 35% O.H. & Profit	=	$ 2.92
Haul - Truck & Driver at $20/hr, 4 per day 20 x 8 ÷ (4 x 290)	=	$ .14
Crane $300 5 man crew $13/hr x 8 x 5 = $520 set 8 per day = 820 ÷ (8 x 290)	=	$ .35
TOTAL COST-IN PLACE	=	$11.74/sq.ft.

Miscellaneous Items

Neoprene O-Ring (installed)	1120 L.F. x $0.40	=	$ 448.00
Shear Connectors	312 units x $1.25	=	390.00
Epoxy Mortar	210 sq.ft. x $0.70	=	147.00
Non-Shrink Grout	400 sq.ft. x $1.20	=	1,680.00
Post-tensioning Complete	1344 L.F. x $4.50	=	6,048.00
	TOTAL COST MISC. ITEMS	=	$8,713.00

Summary

Deck Modules	8000 sq.ft. @ $11.74	=	$93,920.00
Miscellaneous Items		=	8,713.00
	TOTAL COST		$102,633.00

Unit Cost = $102,633.00 ÷ 8000 sq.ft. = $12.83/sq.ft.
= $137.91/sq.m.

ESTIMATED FIELD CONSTRUCTION TIME

Erect Panels	8000 sq.ft. ÷ 290 sq.ft./panel	=	28 panels
	28 panels ÷ 8 panels/day	=	3.5 days
Miscellaneous Work			
Prpearatory work (est)		=	2.0 days
Grouting, Post-tensioning & CIP Concrete (est)		=	7.5 days
	Total Field Time	=	13.0 days

Design Concept 4 - New Construction

Conventional Cast-In-Place Deck Slab

COST ESTIMATE

Concrete (3000 psi) - 309 c.y. @ $240	=	$ 74,160.00
Reinforcing bars - Uncoated - 43,200 Lb @ $.46	=	19,872.00
Reinforcing bars - Epoxy Coated - 35,7000 Lb @ $.70	=	24,990.00
TOTAL COST-IN PLACE	=	$119,022.00

Unit Cost = $119,022.00 : 8000 sq.ft.= $14.88/sq.ft.
= $159.93/sq.m.

TIME AND MANPOWER REQUIREMENTS

Formwork - 5600 sq.ft. ÷ 25 sq.ft./mh 7 man crew - 8 hrs/day = 224 ÷ 7 ÷ 8 = 4 days	=	224 mh
Overhang From Work 1008 sq.ft. ÷ 13 sq.ft./mh 6 man crew - 8 hrs/day = 77.5 ÷ 6 ÷ 8 = 1.6 days	=	77.5 mh
Placing and Typing Reinforcement 79,000 Lbs ÷ 300 Lbs/mh 2 man crew - 8 hr/day = 263 ÷ 2 ÷ 8 = 16.5 days Use 6 man crew for 5.5 days	=	263 mh
Concrete placement 309 c.y. ÷ 1.88 c.y./mh 6 man crew - 8 hrs/day = 164.4 ÷ 6 ÷ 8 = 3.4 days	=	164.4 mh
Concrete finishing - 8000 sq.ft. ÷ 87.5 sq.ft./mh 4 man crew - 8 hrs/day 91.4 ÷ 4 ÷ 8 = 2.9 days	=	91.4 mh
Concrete Curing - 890 sq.yd. ÷ 17.5 sq.yd./mh 4 man crew - 8 hrs/day 50.8 ÷ 4 ÷ 8 = 1.6 days	=	50.8 mh
Form Removal - 1000 sq.ft. ÷ 31 sq.ft./mh 6 man crew - 8 hrs/day 32.3 ÷ 6 ÷ 8 = .7 days	=	32.3 mh

Total Curing time - 14 days

Total days required = 33.7

Assume some overlap of work. Total down time = 26 days

Minimum crew required - 6 men

Maximum crew required - 14 men

* U.S. GOVERNMENT PRINTING OFFICE: 1982 361-428/2034

BEST DOCUMENT AVAILABLE

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

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* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.